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GUIDELINES FOR DETAILING OF STEEL BRIDGES

INDIAN ROADS CONGRESS
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1 INTRODUCTION

The objective of this guideline is to establish and to follow a standard procedure and practice for design and construction of steel road bridges in India. The detailing concept has been developed considering good industry practices, safety norms, erection methodology, shop and site working conditions. This document is meant to serve as the guidelines to design and construction engineers, however compliance with the rules therein does not relieve them in anyway of their responsibility for stability and soundness of the structures designed and erected by them. The work of preparation of this guideline was taken up by B-5 Committee in its tenure 2021-23.

Accordingly, the B-5 Committee constituted a Sub Group (B-5.2) for the preparation of this guideline comprising of Prof. Achyut Ghosh as Convenor & Shri Alok Bhowmick, Prof. Aritra Chatterjee (IIT Kharagpur), Dr. S.K. Garg, Shri A. Ghoshal, Dr. Arul Jayachandran, Shri V. Karthik, Shri Gyana R. Mohanty, Dr. Harshavardhan Subbarao and Shri Anirban Sengupta as members.

The initial draft document was discussed in a number of meetings of the Sub Group and was simultaneously discussed in the meetings of B-5 Committee for necessary modifications. The draft document after going through various modifications was discussed at length and finalized in the 14th meeting of B-5 Committee held on 28th October, 2023

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The draft document was recommended by the Technical Standing Panel comprising of Shri A. Ghoshal, Shri V.N. Heggade and Dr. S.K. Garg, in its meeting held on 14th November, 2023. The finalized draft was placed before the Bridges Specifications & Standards Committee (BSS) in its meeting held on 18th November, 2023 and was approved for placing before the Executive Committee & IRC Council for their consideration. After due deliberation on the draft document, the Executive Committee in its meeting held on 1st December, 2023 approved the same for placing before the Council. The draft guidelines was approved by the IRC Council in its 226th Meeting held on 2nd December, 2023 at Gandhinagar (Gujarat) for printing.

2 GENERAL

2.1 Scope: The scope of the document is outlined as follows:

2.1.1 This document gives non-contradictory complementary information for use with IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI – Composite Construction (Limit State Design) Third Revision", IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Section V – Steel Road Bridges, (Limit State Method) (Third Revision)" and IRC:SP:104 "Guidelines for Fabrication and Erection of Steel Bridges". The aim of this document is to provide guidance on detailing of typical steel road bridges along with a brief on the force transfer mechanisms.

2.1.2 The design criteria for steel bridges are covered in IRC:24.

2.1.3 The design criteria for composite bridges are covered in IRC:22.

2.1.4 The criteria for fabrication and erection of steel bridges are covered in IRC: SP:104.

2.1.5 Special requirements of seismic design are not covered in this document. Reference should be made to the requirements given in IRC:SP:114 "Guidelines for Seismic Design of Road Bridges".

2.1.6 This document states only the minimum requirements necessary.
Note *- The publication does not purport to include all the necessary provisions in a structure. Users are responsible for its correct application.*

2.1.7 The scope is restricted to detailing concept and design aspect for plate girders bridges and through type, semi through type & deck type Open Web Girder Bridges (truss bridges). This guideline does not cover the requirement for the following type of structure:

i) Tub type composite girder
ii) Single cell or multicell steel box girder
iii) Steel arch girders
iv) Cable supported bridges

2.1.8 The intent of this document is to guide designers for design and detailing of steel bridges with good engineering practices. These provisions are to be considered as recommendatory in nature. In case any of the provisions of this document is in conflict with the provision of code, the provisions of code shall prevail. Sample drawings, Connection detailing as shown in this document are only indicative and for guidance. Designers are free to use any other alternative detail which satisfies the intended purpose.

2.2 Limitations & Exclusions

The guidelines mainly aims to address the detailing requirement of typical steel road bridges along with a brief of the general force transfer mechanism at different types of joints & splices. The limitations mentioned in Clause 501.2 provided in IRC:24 is also applicable for this code. It does not provide a detailed insight on the design parameters and design methodology for which needs to be referred to IRC:24 & IRC:22 as applicable.

Some typical methodology for erection has been covered as **Appendix – 1**. Erection methodology suitable for the site condition and considering time cycle may adopted by the designer and accordingly all detailing and design guideline provided may be modified. However, the fabrication & erection procedures shall be as per IRC:SP:104.

2.3 Standard General Notes for Steel Bridge Works

The typical general notes for steel constructed road bridges have been enclosed in **Appendix – 2**. These notes are for general guideline. Any change if required for specific application can be adopted as per IRC: 22, IRC: 24, IRC:SP:104, IRC:SP:114, IS: 9595, IS: 4000, IS: 1367 (all parts), IS: 814 or

as per requirement of contract or design requirement or as per any acceptable National / International standard.

2.4 Do's & Don'ts for Detailing

2.4.1 General

i) Verification of length for all inclined members and gusset plates need to be performed at shop prior to cutting the sections and plates as per the drawing.
ii) The material strength test and elongation have to be carried out.
iii) Prior to sending the fabricated material at site for erection, a trial assembly should be made at fabrication yard for checking and if required necessary corrections need to be carried out.
iv) Camber check need to be performed at shop on trial assembly at shop prior to transportation to site the fabricated material.
v) All workmen should use proper protective gear as per guideline.
vi) Quality Assurance plan for fabrication procedure need to be prepared, preapproved and to be followed.
vii) Group similar connections rather than have several different connections. Connections on a project should be as uniform as possible to save fabrication time and reduce the possibility of errors.

2.4.2 Welding

i) The sequence of welding needs to be followed as per instructions in drawings.
ii) The procedure for welding especially the return run for welding, back gouging etc. as applicable has to be provided as per provision in IRC:SP:104.
iii) All welding has to be tested as per provision of IRC:SP:104.
iv) During welding of thick plates more than 20 mm precaution to avoid warping due to temperature gradient need to be taken care by preheating as per provision of IS: 9595.
v) Welding Procedure Specification need to be prepared as per provisions of IS: 9595 & IS: 4353 & IRC:SP:104, preapproved and to be followed.
vi) Avoid overhead welding.
vii) Provide proper clearance for bolted and welded connections.

2.4.3 Bolts

i) Torque in HSFG bolt to be applied based on Slip factor and Tension required and manufacturer's recommendation with prior approval of Designer/Engineer-in-Charge.
ii) All bolt tightening must be checked using filler gauge as per provision of IS: 4000 & IRC:SP:104 and by hammering the tightness.

iii) Permanent bolts need to be tightened finally or final torque to be applied after completing the alignment check after completion of erection. Till such time the bridge needs to be kept erected with 50% bolt snug tightened along with 50% drift.
iv) Limit the number of bolt diameters as far as possible, preferably one (a maximum of three sizes if necessary).
v) Avoid different grade bolts with the same diameter.
vi) Specify slip-critical bolts only when necessary. If slip-critical joints and bolts are needed, the steel detailer must indicate them on the erection and shop drawings.
vii) The structure needs to be checked for safety at erection condition with 50% HT bolt and 50% drift in place of 100% HSFG final bolt at joints.

3 CHARACTERISTICS OF BOLTS AND WELDS

There are two common ways to connect structural steel members - using bolts or welds. Rivets, while still available, are not currently used for new structures and will not be considered here. This chapter will presentthe basic properties and requirements for bolts and welds.

3.1 Bolts

A bolt is a mechanical fastener with a shaft having male threaded matching with female thread such as nut.

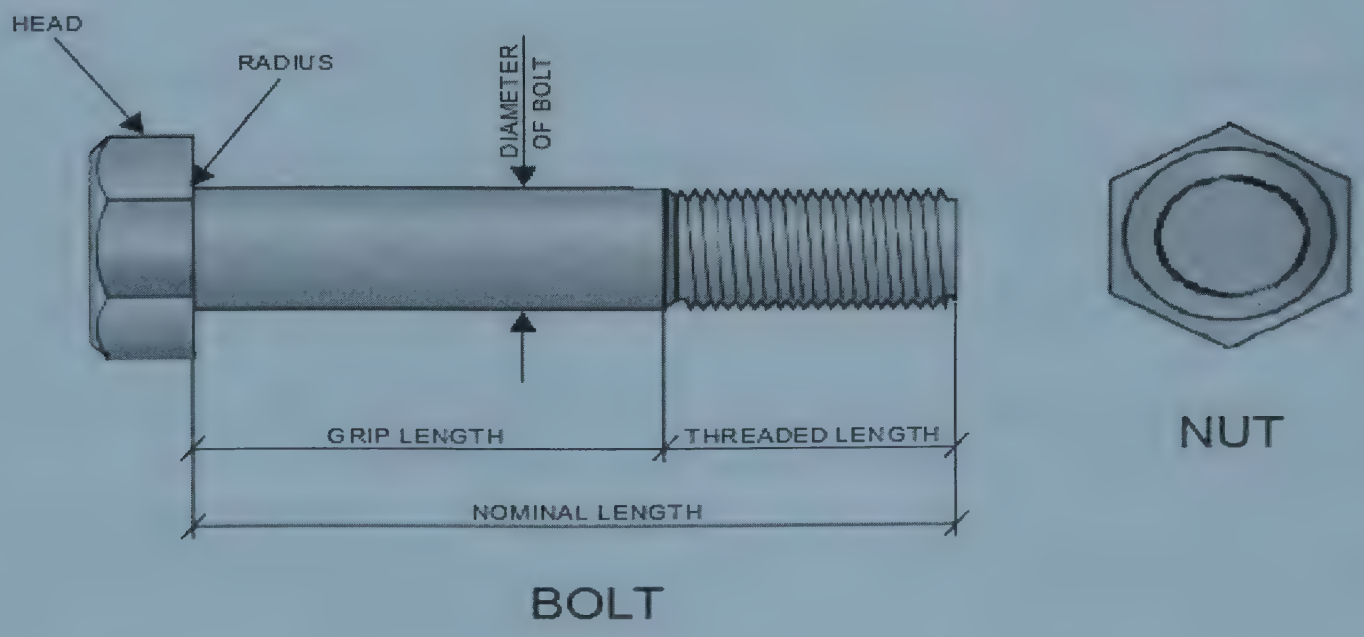


Fig. 1 Bolt and Nut

Bolt is a non-tapered fastener that is generally used with washer & nuts to hold the object together. The thread of a bolt is cylindrical.

Bolt Head – There are many types of Bolt heads such as:

i) Flat Bolt Head – This Head is counter shank with a flat top.

ii) Hex Bolt Head – It is a Hexagonal Head.
iii) Round bolt Head – Dome Type Head etc.

There are many types of bolts such as:

i) **Anchor Bolt:** Fasteners that secure the framing of a structure to the foundation.

ii) **Blind Bolt:** A blind bolt consists of three things, a steel pin, a collar and a sleeve. When tightening the blind bolt, the collar creates a pressure that collapses the expander on the blindside while the sleeve collapses to create a seal in the visible side of the blind bolt.

iii) **Carriage Bolt:** Carriage bolts are an application specific fastener designed for use in carpentry, wood base materials and the construction of wood products.

iv) **Double End Bolt:** Double end bolts are externally threaded fasteners without a head, used with 2 nuts on either side.

v) **Eye Bolt:** An eye bolt is a bolt with a loop at one end. They are used to firmly attach a securing eye to a structure, so that ropes or cables may then be tied to it.

vi) **Flange Bolt:** Flange bolts have a circular flange under the head that acts like a washer to distribute the load.

vii) **Hex Bolt:** Hex bolts are bolt fasteners with six sided hexagon head and externally threaded body.

viii) **T-Headed Bolt:** A bolt with a T-shaped head, made to fit into a T-shaped slot.

ix) **U-Bolts:** A U-bolt is a u-shaped curved bolt having threads on each end mainly used as support in the piping and pipeline.

Also, Bolts are sometime classified as:

i) **Clearance Tolerance Bolts (Black Bolts):** Black bolts are of mild steel, but while tightening too much the bolts twist off easily. These are not used in Bridges.

ii) **Turned & Fitted Bolts:** Turned & fitted bolts having uniform shank diameter and inserted in close tolerance of the holes. These are also called precision bolts.

iii) **High Strength Friction Grip (HSFG) Bolts:** HSFG Bolts are used in connection which transfers load through friction.

Nuts

Nuts are used along with bolts as fastening device, outside shape of nuts are mainly hexagonal & inside is circular with thread to match with bolt.

Washer

Washer is used in conjunction with fasteners to distribute the land from nut & bolts over a larger area. There are different type of washers used in the connections:

i) Plain Washer
ii) Spring Washer
iii) Lock Washer
iv) Direct Tension Indicator (DTI) Washer

For DTI washer dimensions can be referred to EN14399-9.

3.1.1 Material

Steel is the most used material about (90%) in fastener production. Other materials like alloy steel, silicon, bronze, stainless steel, aluminum, brass & chrome are also used in bolt.

3.1.1.1 Grade of Steel for Bolts: Property class of steel bolt are designated with their tensile strength (N/mm^2) & yield stress (N/mm^2).

Fig. 2 Bolt Head showing Bolt Grade

Grade of a bolts is designated as X, Y. In this nomenclature, first part X indicates the minimum ultimate tensile strength of the bolt in T/cm^2 and the second part Y indicates the ratio of the yield stress to ultimate stress as fraction, e.g., grade 10.9 bolt will have a minimum ultimate strength 10 T/cm^2 (i.e., 1000 N/mm^2) and minimum yield strength of (0.9 x 10) = 9 T/cm^2 (i.e., 900 N/mm^2).

Table 1 Strength of Bolt - Grade Designation

Strength Grade Designation	4.6	6.8	8.8	10.9	12.9
Min. Tensile Strength(N/mm^2)	400	600	800	1000	1200
Yield Strength(N/mm^2)	240	480	-	-	-
Stress at 0.2% Nonproportional Elongation $R_{p0.2}$ (N/mm^2)	-	-	640	900	1080

*Table is correct for g = 10 m/s^2

Turned and fitted bolts are available from grade 4.6 to grade 8.8. Turned-and-fitted bolts have uniform shanks and are inserted in close tolerance drilled holes and made snug tight by box spanners. For the purpose of HSFG connections, only high strength structural bolts of two property classes: 8.8 and 10.9 can be used.

3.1.1.2 Grade of Steel for Nuts

Table 2 Strength of Nut - Grade Designation

Strength Grade Designation	**4**	**6**	**8**	**10**	**12**
Min. Tensile Strength(N/mm^2)	400	600	800	1000	1200

Nut should be used one class higher than that of bolt. For 12.9 grade bolts, nuts of same grade may be used.

Lock nut – this is a nut which prevents loosening the same under vibration.

3.1.2 Basic Profile of Thread of Bolt (as per IS: 4218:2001)

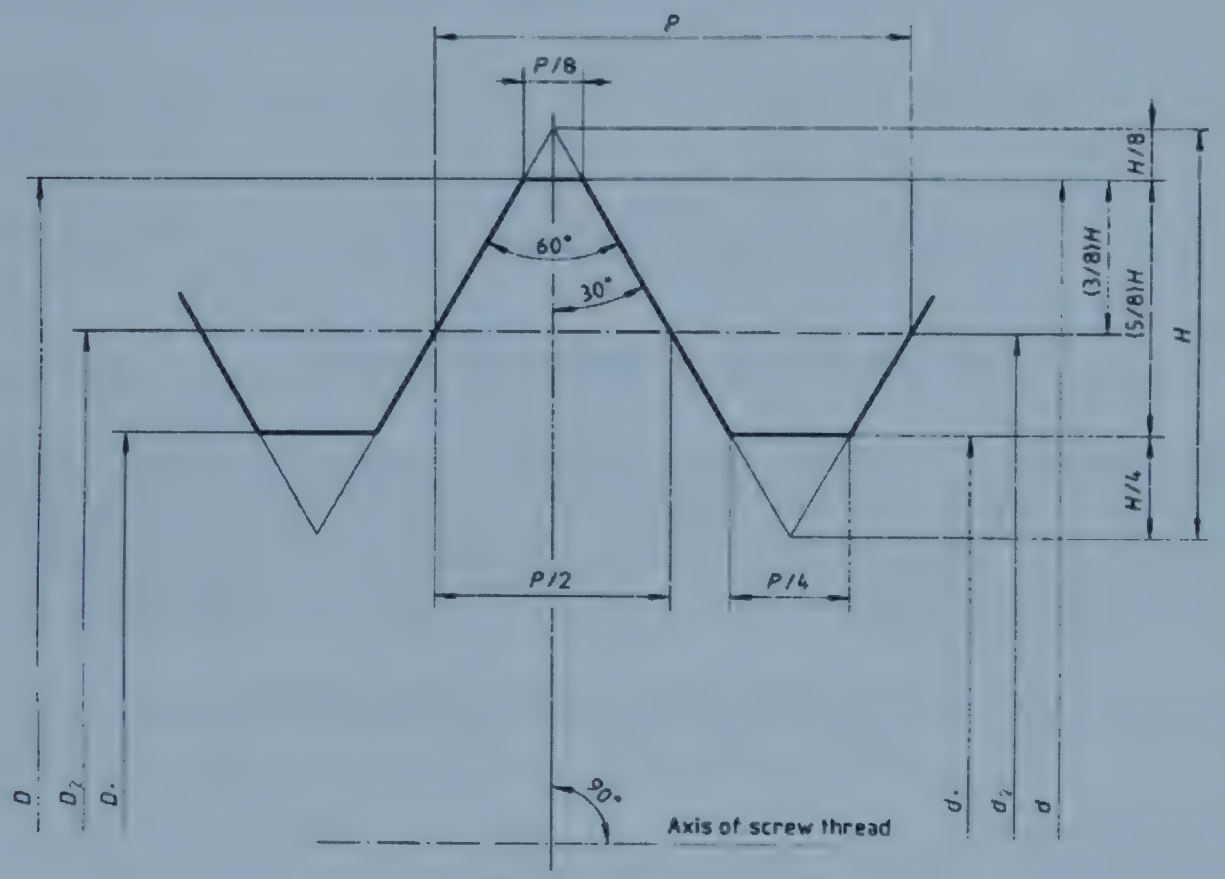


Fig. 3 Basic Profile of Thread

D	:	Basic major diameter of internal thread
d	:	Basic major diameter of external thread (nominal diameter)
D_2	:	Basic pitch diameter of internal thread.
d_2	:	Basic pitch diameter of external thread.
D_1	:	Basic minor diameter of internal thread
d_1	:	Basic minor diameter of external thread
H	:	Height of fundamental Triangle, $H = \sqrt{3}/2\,P$
P	:	Pitch

3.1.3 Determination of Stress Area

$$A_n = \frac{\pi}{4}\left(\frac{d_2 + d_3}{2}\right)^2$$

Where,

A_n = Net tensile area

d_2 = Basic pitch diameter of external thread

d_3 = $d_1 - {}^{H}/_{6}$

H = ${}^{\sqrt{3}}/_{2}P$

For d_1, d_2 and H refer **Fig. 3.**

For value of pitch P (**coarse**), refer **Table 1** of IS: 4218 (Part 4): 2001. This has been prepared considering coarse pitch, for other pitches values will be different.

For value of d_1 and d_2, refer **Table 1** of IS: 4218 (Part 3): 1999.

A_n for commonly used bolts are shown in **Table 3.**

Table 3 Shank and Net Area for Bolts

Bolt	Nominal Bolt Diameter (mm)	Pitch P (coarse) (mm)	*H (m)*	d_1 *(mm)*	d_2 *(mm)*	d_3 *(mm)*	A_s *(mm^2)*	A_n *(mm^2)*
M12	12	1.75	1.516	10.106	10.863	9.853	113.1	84.3
M16	16	2	1.732	13.835	14.701	13.546	201.1	156.7
M18	18	2.5	2.165	15.294	16.376	14.933	254.5	192.5
M20	20	2.5	2.165	17.294	18.376	16.933	314.1	244.8
M22	22	2.5	2.165	19.294	20.376	18.933	380.1	303.4
M24	24	3	2.598	20.752	22.051	20.319	452.4	352.5
M27	27	3	2.598	23.752	25.051	23.319	572.6	459.4
M30	30	3.5	3.031	26.211	27.727	25.706	706.9	560.6
M33	33	3.5	3.031	29.211	30.727	28.706	855.3	693.6
M36	36	4	3.464	31.67	33.402	31.093	1017.9	816.7
M42	42	4.5	3.897	37.129	39.077	36.48	1385.4	1120.9
M48	48	5	4.33	42.587	44.752	41.865	1809.5	1473.1

A_s = Shank area of bolt

3.1.4 Diameter of Bolt Hole

i) **Standard Clearance Hole:** Diameter for bolt holes are mentioned in Clause 512.5.4 (a) of IRC:24.

ii) **Over Size Hole:** Holes of size larger than the standard clearance holes may be used in slip resistant connections and hold down bolted connections, only where specified, provided the oversize holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole not larger than the standard clearance hole (and hardened washer in slip resistant connections).

Oversize hole shall not exceed $1.25d$ or $(d + 8)$ mm in diameter, where d is the nominal bolt diameter, in mm.

Diameter of bolt hole for HSFG bolt is mentioned in Clause no 1.11 of IRC:SP:104.

iii) **Short and Long Slots:** Slotted holes of size larger than the standard clearance hole may be used in slip resistant connections and hold down bolted connections, only where specified, provided the oversize holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole of size not larger than the standard clearance hole (and hardened washer in slip resistant connection). A short-slotted hole shall not exceed the appropriate hole size in width and $1.33d$ in length. A long-slotted hole shall not exceed the appropriate hole size in width and $2.5d$ in length. If the slot length is larger than those specified, shear transfer in the direction of slot is not admissible even in friction type of connection.

iv) **Fitted Bolt Holes:** Holes for turned and fitted bolts shall be drilled to a diameter equal to the nominal diameter of the shank or barrel subject to tolerance specified in IS: 919 (Parts 1 and 2). Diameter of bolt hole for turned and fitted bolt is also mentioned in Clause 1.11 of IRC:SP:104.

3.1.5 Edge Distance

Edge distance for bolt holes is mentioned in Clause 512.5.4 (b) of IRC:24.

3.1.6 Pitch of Bolts

Pitch of bolts are mentioned in Clause 512.5.4 (c) of IRC:24. Maximum pitch distances are shown in **Fig. 4**. Minimum pitch for bolts is shown in **Table 4** and **Table 5**. Gauge distances for common rolled sections are shown in **Fig. 5** and **Table 6**.

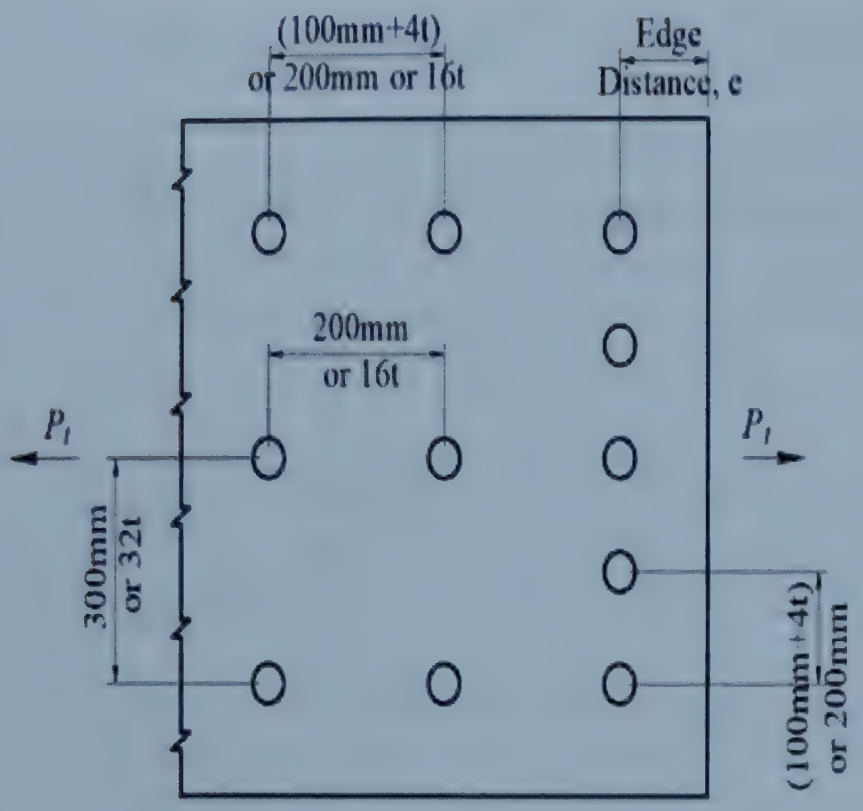
Gauge distances for common rolled sections are shown in **Fig. 5** and **Table 6**.

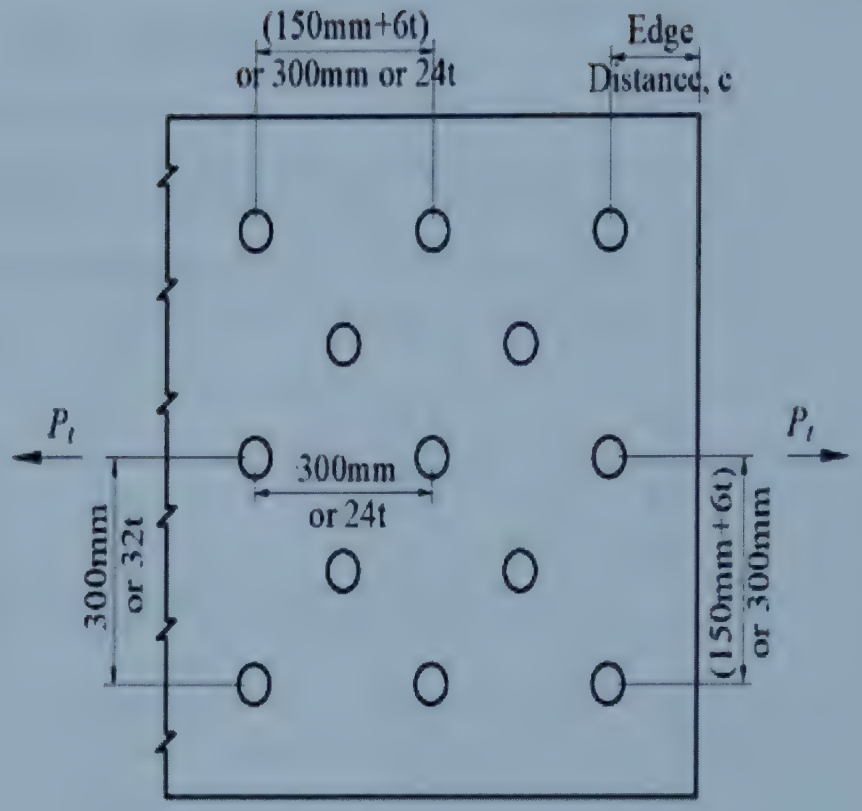
Table 4 Hole Diameter, Minimum Edge Distance, Minimum Spacing for Black Bolts

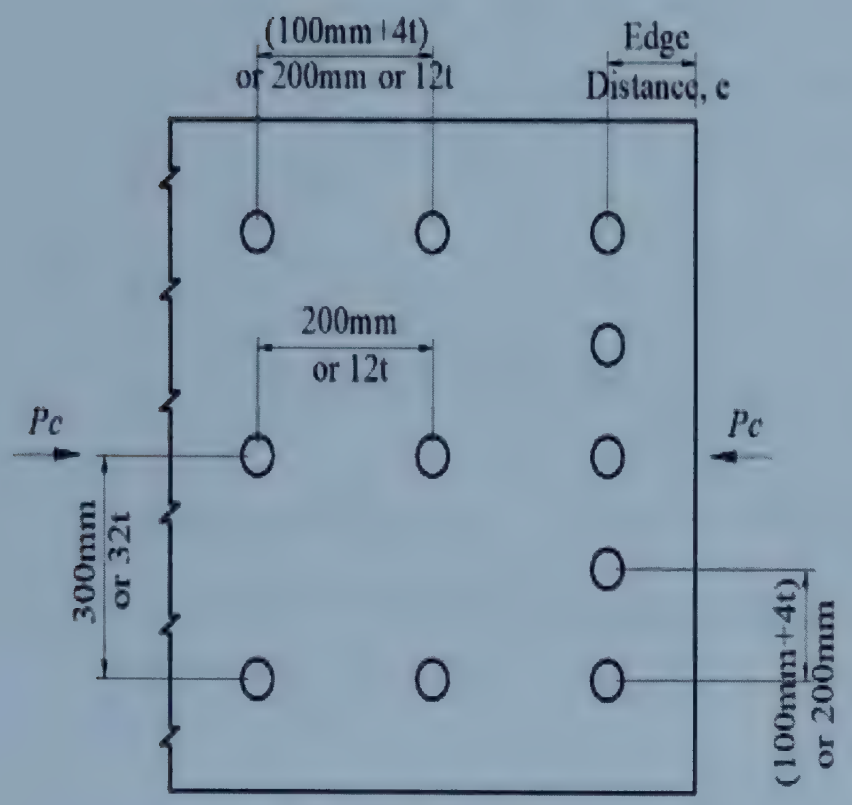
Diameter of bolt (mm)	Diameter of hole of bolt (mm)	Minimum edge distance, *e* (mm)		Minimum distance between centers of any adjacent bolts, *p* (mm)
		Rolled, machine flame cut, swan or plane edge	Sheared, hand flame cut edge	
12	13.5	21	24	30
16	17.5	27	31	40
18	19.5	30	35	45
20	21.5	33	38	50
22	23.5	36	42	55
24	25.5	39	45	60
27	29	44	51	68
30	32	48	56	75
33	35	53	62	83
36	38	57	67	90
42	44	66	77	105
48	50	75	88	120

Table 5 Hole Diameter, Minimum Edge Distance, Minimum Spacing for HSFG Bolts (IRC: SP:104)

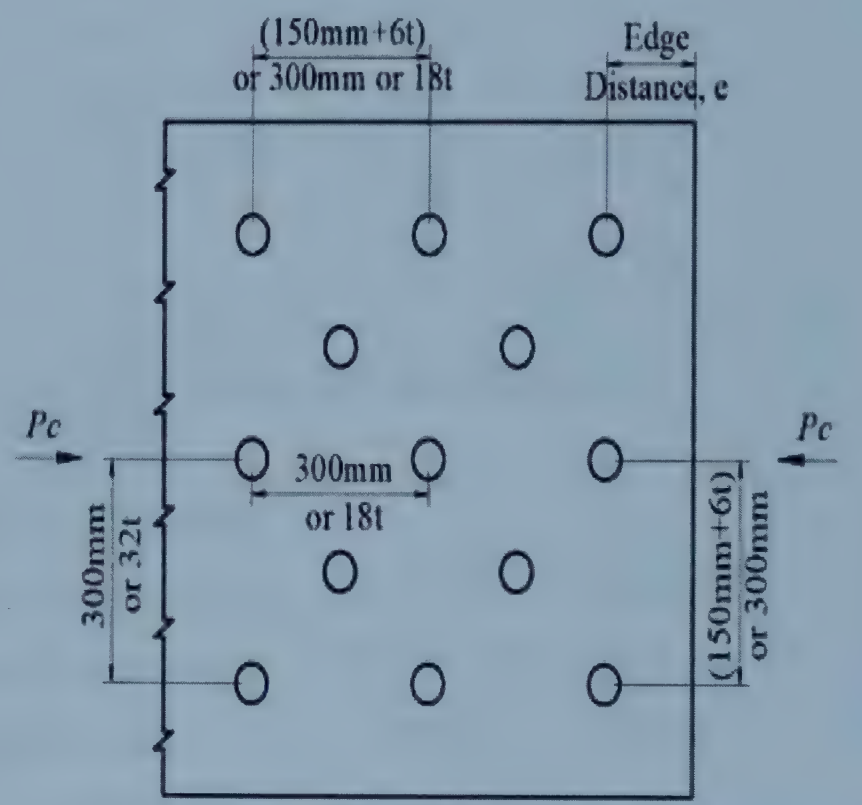
Diameter of bolt (mm)	Diameter of hole of bolt (mm)	Minimum edge distance, *e* (mm)		Minimum distance between centers of any adjacent bolts, *p* (mm)
		Rolled, machine flame cut, swan or plane edge	Sheared, hand flame cut edge	
16	18	27	32	40
20	22	33	39	50
24	26	39	46	60
30	32	48	56	75
36	38	57	67	90
42	44	66	77	105







Bolts in Normal Spacing



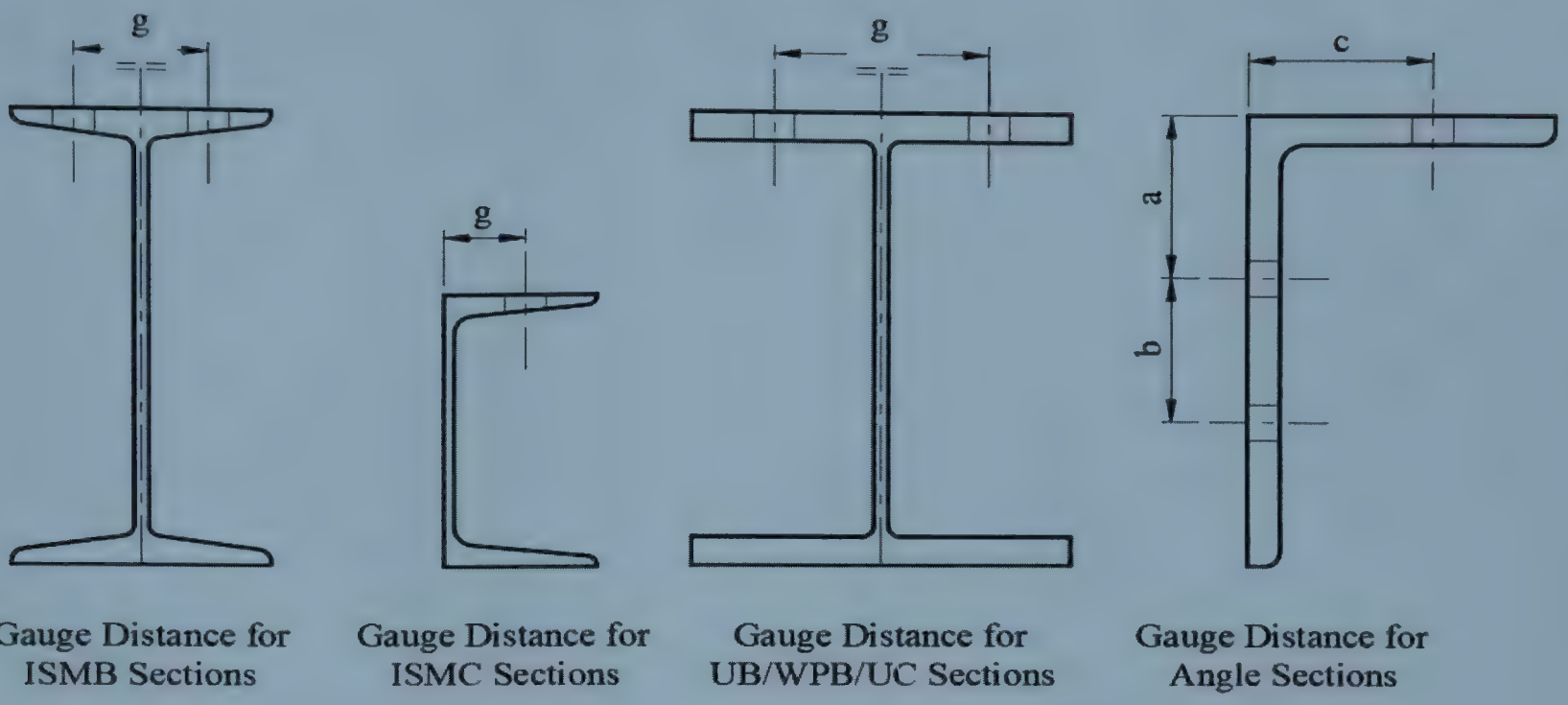
Bolts in Staggered Spacing

Fig. 4 Maximum Pitch of Bolts

Note:

a) Spacing of the bolts should be the least relevant value.

b) For bolts in staggered spacing, the figure has been prepared considering maximum gauge of 75 mm.



Gauge Distance for ISMB Sections | Gauge Distance for ISMC Sections | Gauge Distance for UB/WPB/UC Sections | Gauge Distance for Angle Sections

Fig. 5 Gauge Distance for Rolled Section

Table 6a: ISMB Sections

Section	Minimum g (mm)	Maximum diameter of Bolt (mm)
ISMB 125	35	12
ISMB 150	40	12
ISMB 200	55	16
ISMB 250	65	22
ISMB 300	80	22
ISMB 350	80	22
ISMB 400	80	22
ISMB 450	90	22
ISMB 500	100	28
ISMB 550	100	32
ISMB 600	140	25
ISMB 600	100	32

Table 6b: ISMC Sections

Section	Minimum g (mm)	Maximum diameter of Bolt (mm)
ISMC 75	21	12
ISMC 100	28	16
ISMC 125	35	22
ISMC 150	40	25
ISMC 175	40	25
ISMC 200	40	25
ISMC 225	45	25
ISMC 250	45	25
ISMC 300	50	28
ISMC 400	60	28

Table 6c: WPB Sections

Section	Minimum *g* (mm)	Maximum *g* (mm)
WPB 600 x 300 x 129	116	210
WPB 600 x 300 x 178	117	210
WPB 600 x 300 x 212	120	210
WPB 600 x 300 x 285	125	215
WPB 700 x 300 x 150	117	210
WPB 700 x 300 x 204	118	210
WPB 700 x 300 x 241	121	210
WPB 700 x 300 x 301	125	214

Table 6d: UB Sections

Section	Minimum *g* (mm)	Maximum *g* (mm)
UB 203 x 133 x 25	66	79
UB 203 x 133 x 30	66	80
UB 254 x 146 x 31	76	80
UB 254 x 146 x 37	76	80
UB 254 x 146 x 43	77	81
UB 305 x 165 x 40	86	87
UB 305 x 165 x 46	87	88
UB 305 x 165 x 54	88	89
UB 406 x 178 x 54	88	100
UB 406 x 178 x 60	88	100
UB 406 x 178 x 67	89	101
UB 406 x 178 x 74	90	102
UB 610 x 229 x 101	101	138
UB 610 x 229 x 113	101	138
UB 610 x 229 x 125	102	139
UB 610 x 229 x 140	103	140

Table 6e: UC Sections

Section	Minimum *g* (mm)	Maximum *g* (mm)
UC 152 x 152 x 23	76	86
UC 152 x 152 x 30	77	87
UC 152 x 152 x 37	78	88
UC 203 x 203 x 46	97	114
UC 203 x 203 x 52	98	114
UC 203 x 203 x 60	99	116
UC 203 x 203 x 71	100	116
UC 203 x 203 x 86	103	119
UC 254 x 254 x 73	99	165
UC 254 x 254 x 89	100	166
UC 254 x 254 x 107	103	169
UC 254 x 254 x 132	105	171
UC 254 x 254 x 167	109	175
UC 305 x 305 x 97	100	215
UC 305 x 305 x 118	102	217
UC 305 x 305 x 137	104	219
UC 305 x 305 x 158	106	221
UC 305 x 305 x 198	109	225
UC 305 x 305 x 240	113	228
UC 305 x 305 x 283	117	232

Table 6f: Bolt Gauge Distances in Legs of Angles

Leg Size (mm)	Double row of bolt		Single row of bolts	Maximum bolt size for double row of bolts (mm)
	a (mm)	b (mm)	c (mm)	
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	-	-	55	-
90	-	-	50	-
80	-	-	45	-
75	-	-	40	-
70	-	-	40	-
65	-	-	35	-
60	-	-	35	-
55	-	-	30	-
50	-	-	28	-
45	-	-	25	-
40	-	-	21	-
35	-	-	19	-
30	-	-	17	-
25	-	-	15	-
20	-	-	12	-

3.1.7 Action of Bolts

When bolts are tightened, the joint pushes outward on the bolt, keeping the bolt in tension. Due to the tension in bolt, the joint will deflect as shown in **Fig. 6a**. After tightening of the bolt, when external tensile load is applied on the bolt, it relieves the joint. Since strain is proportional to stress, the partially relieved joint partial return to the original thickness as shown in **Fig. 6b**.

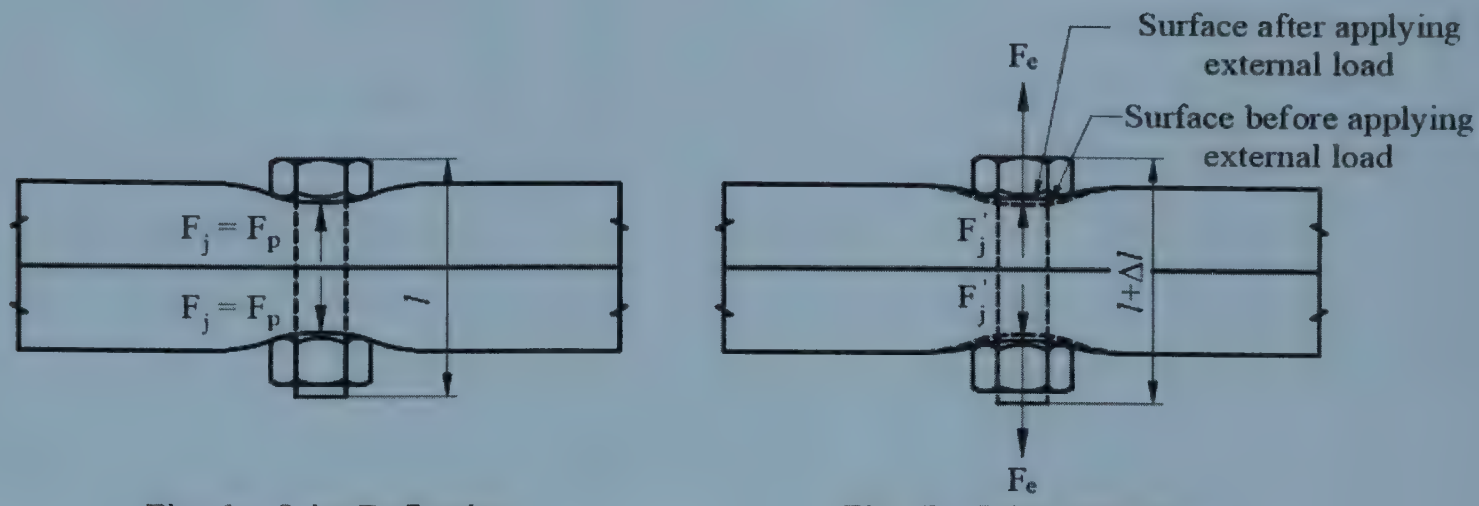


Fig. 6a : Joint Deflection after tightening of bolt

Fig. 6b : Joint Deflection after applying external load

Fig. 6 Joint Deflection

The bolt, under the action of the combined joint force and external force, will also elongate. Increase in length in the bolt will be equal to the increase in thickness in the joint. As stiffness of the bolt is less than that of the joint, the change in load in the bolt will be lesser than the change of the load in the joint. Hence, when external tensile load is applied in a bolted joint, the increase in force acting on the bolt is only a small portion of this external load; the rest of the external load is absorbed by the joint.

When Bolt is elongated due to tensile load, its elongation may be expressed as:

$$Elongation\ of\ bolt = \frac{Length\ of\ bolt \times Load\ in\ bolt}{Net\ area\ of\ bolt \times E} - Deformation\ in\ thread\ due\ to\ transfer\ of\ force$$

3.1.8 Strength of Bolts other than Friction Grip Bolts

The ultimate strengths given for bolts and rivets in shear and bearing apply only to bolts and rivets in holes not larger than the sizes given in **Table 3**.

Black bolts should not be used in permanent main structural connections of highway bridges.

For fasteners other than HSFG bolts, the ultimate limit state of a connection should be deemed to be reached when the design load on any fastener equals its ultimate capacity, determined in accordance with Clause 3.1.8.1, 3.1.8.2 and 3.1.8.3.

3.1.8.1 Tension Capacity of Bolt (Refer Clause 512.5.5.5 of IRC:24)

The tension capacity of bolts has been shown in **Table 7**.

Table 7 Tension Capacity of Bolt

Diameter of Bolt (mm)	A_{sb} (mm^2)	A_n (mm^2)	Tension Capacity (kN)				
			For Grade of Bolt				
			4.6	6.8	8.8	10.9	12.9
12	113.1	84.3	24.3	36.4	48.6	60.7	72.8
16	201.1	156.7	43.9	67.7	90.3	112.8	135.4
18	254.5	192.5	55.4	83.2	110.9	138.6	166.3
20	314.2	244.8	68.6	105.8	141.0	176.3	211.5
22	380.1	303.4	82.9	131.1	174.8	218.4	262.1
24	452.4	352.5	98.7	152.3	203.0	253.8	304.6
27	572.6	459.4	124.9	198.5	264.6	330.8	396.9
30	706.9	560.6	154.2	242.2	322.9	403.6	484.4
33	855.3	693.6	186.6	299.6	399.5	499.4	599.3
36	1017.9	816.7	222.1	352.8	470.4	588.0	705.6
42	1385.4	1120.9	302.3	484.2	645.6	807.0	968.5
48	1809.6	1473.1	394.8	636.4	848.5	1060.6	1272.8

3.1.8.2 Shear Capacity of Bolt (Refer Clause 512.5.5.3 of IRC:24)

The design shear strength of the bolt for some typical cases is shown in **Table 8** and **Table 9**.

Table 8 Shear Capacity of Bolts with $n_s = 1$, $n_n = 0$

d (mm)	A_{sb} (mm^2)	Shear Capacity (kN)				
		For Grade of Bolt				
		4.6	6.8	8.8	10.9	12.9
12	113.1	20.9	31.3	41.8	52.2	62.7
16	201.1	37.2	55.7	74.3	92.9	111.5
18	254.5	47.0	70.5	94.0	117.5	141.1
20	314.2	58.0	87.1	116.1	145.1	174.1
22	380.1	70.2	105.3	140.4	175.6	210.7
24	452.4	83.6	125.4	167.2	209.0	250.7
27	572.6	105.8	158.7	211.6	264.5	317.4
30	706.9	130.6	195.9	261.2	326.5	391.8
33	855.3	158.0	237.0	316.0	395.0	474.1
36	1017.9	188.1	282.1	376.1	470.1	564.2
42	1385.4	256.0	383.9	511.9	639.9	767.9
48	1809.6	334.3	501.5	668.7	835.8	1003.0

Table 9 Shear Capacity of Bolts with $n_s = 1$, $n_n = 0$

d (mm)	A_n (mm²)	Shear Capacity (kN) For Grade of Bolt				
		4.6	**6.8**	**8.8**	**10.9**	**12.9**
12	84.3	15.6	23.4	31.1	38.9	46.7
16	156.7	29.0	43.4	57.9	72.4	86.9
18	192.5	35.6	53.3	71.1	88.9	106.7
20	244.8	45.2	67.8	90.5	113.1	135.7
22	303.4	56.1	84.1	112.1	140.1	168.2
24	352.5	65.1	97.7	130.3	162.8	195.4
27	459.4	84.9	127.3	169.8	212.2	254.6
30	560.6	103.6	155.4	207.1	258.9	310.7
33	693.6	128.1	192.2	256.3	320.4	384.4
36	816.7	150.9	226.3	301.8	377.2	452.7
42	1120.9	207.1	310.6	414.2	517.7	621.3
48	1473.1	272.2	408.2	544.3	680.4	816.5

The above shear capacity should be multiplied by following reduction factors as applicable:

i) **Reduction Factor for Long Joints (Refer Clause 512.5.8 of IRC:24)**

When the length of the joint, lj of a splice or end connection in a compression or tension element containing more than two bolts (that is the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds $15d$ in the direction of load, the shear capacity should be reduced by reduction factor for long joints, βl_j, given by:

$$\beta_{lj} = 1.075 - {}^{l_j}/_{200d} \quad but\ 0.75 \leq \beta_{lj} \leq 1$$

Where,

lj = Length of the joint of a slice or end connection of a compression or tension element containing more than two bolts exceeds $15d$ in the direction of load.

ii) **Reduction Factor for Large Grip Length (Refer Clause 512.5.9 of IRC:24)**

When the grip length, l_g (equal to the total thickness of the connected plates) exceeds $5d$, the design shear capacity shall be reduced by a factor βl_g, given by:

$$\beta_{lg} = 8d/(3d + l_g)$$

β_{lg} shall not be more than β_{lj}, as given earlier.

The grip length, l_g shall in no case be greater than $8d$.

For example, suppose 3 nos. 32 mm thick plates are connected using 16 mm diameter bolts.

$$l_g = 3 \times 32 = 96\ mm > 5d = 80\ mm\ (8 \times 16)$$

$$\beta_{lg} = {}^{(8 \times 16)}/(3 \times 16 + 96) = 0.89$$

iii) **Reduction Factor for Packing Plates (Refer Clause 512.5.9 of IRC:24)**

The design shear capacity of bolts carrying shear through packing plates in excess of 6 mm shall be decreased by a factor β_{pk}, given by:

$$\beta_{pk} = (1 - 0.0125 t_{pk})$$

t_{pk} = Thickness of packing plate

For example, suppose 12 mm plate is used as packing plate in a connection,

$$t_{pk} = 12\ mm$$

$$\beta_{pk} = (1 - 0.0125 \times 12) = 0.85$$

3.1.8.3 Bearing Capacity of the Bolt (Refer Clause 512.5.5.4 of IRC:24)

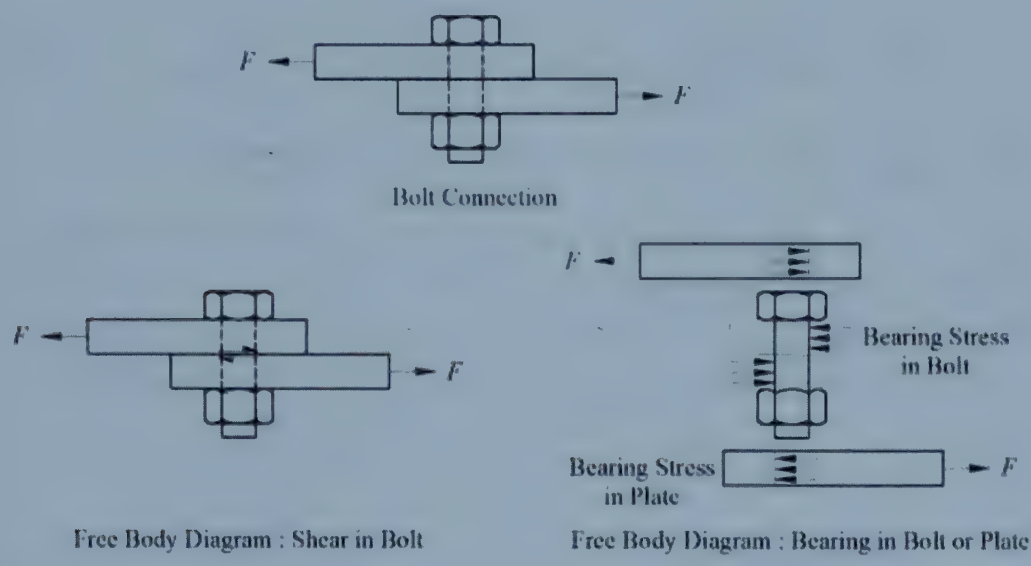


Fig. 7 Bolts in Shear and Bearing

Table 10 Values of f_{ub}/f_u

Grade of Bolt	Grade of Plate		
	E250	E350	E410
4.6	0.98	0.82	0.74
6.8	1.46	1.22	1.11
8.8	1.95	1.63	1.48
10.9	2.44	2.04	1.85
12.9	2.93	2.45	2.22

As the bearing capacity of a bolt depends upon thickness of the connecting plates, values bearing capacity per mm thickness of plate have been shown in tables below. Minimum edge distance and pitch as specified in **Table 4** and **Table 5** are used in calculation of bearing strength as shown in **Table 11, Table 12, Table 13** and **Table 14**.

Table 11 Bearing Capacity for Black Bolts per mm Thickness of Plate (Applicable for Grade of Bolt - 4.6)

d (mm)	d_0 (mm)	$e/3d_0$	$p/3d_0 - 0.25$	k_b	Bearing Capacity, V_{dpb} (kN) per mm Thickness of Plate of Grade		
					E250	E350	E410
12	13.5	0.52	0.49	0.49	4.70	4.70	4.70
16	17.5	0.51	0.51	0.51	6.53	6.53	6.53
18	19.5	0.51	0.52	0.51	7.34	7.34	7.34
20	21.5	0.51	0.53	0.51	8.16	8.16	8.16
22	23.5	0.51	0.53	0.51	8.98	8.98	8.98
24	25.5	0.51	0.53	0.51	9.79	9.79	9.79
27	29	0.51	0.53	0.51	11.02	11.02	11.02
30	32	0.50	0.53	0.50	12.00	12.00	12.00
33	35	0.50	0.54	0.50	13.20	13.20	13.20
36	38	0.50	0.54	0.50	14.40	14.40	14.40
42	44	0.50	0.55	0.50	16.80	16.80	16.80
48	50	0.50	0.55	0.50	19.20	19.20	19.20

Table 12 Bearing Capacity for High Strength Bolts per mm Thickness of Plate (Applicable for Grade of Bolt - 4.6)

d (mm)	d_0 (mm)	$e/3d_0$	$p/3d_0 - 0.25$	k_b	Bearing Capacity, V_{dpb} (kN) per mm Thickness of Plate of Grade		
					E250	E350	E410
16	18	0.50	0.49	0.49	6.27	6.27	6.27
20	22	0.50	0.51	0.50	8.00	8.00	8.00
24	26	0.50	0.52	0.50	9.60	9.60	9.60
30	33	0.51	0.51	0.51	12.24	12.24	12.24
36	39	0.50	0.52	0.50	14.40	14.40	14.40
42	45	0.50	0.53	0.50	16.80	16.80	16.80

Table 13 Bearing Capacity for Black Bolts per mm Thickness of Plate (Applicable for Grade of Bolt - 6.8, 8.8, 10.9, & 12.9)

d (mm)	d_0 (mm)	$e/3d_0$	$p/3d_0 - 0.25$	k_b	Bearing Capacity, V_{dpb} (kN) per mm Thickness of Plate of Grade		
					E250	E350	E410
12	13.5	0.52	0.49	0.49	4.82	5.76	6.35
16	17.5	0.51	0.51	0.51	6.69	8.00	8.81
18	19.5	0.51	0.52	0.51	7.53	9.00	9.91
20	21.5	0.51	0.53	0.51	8.36	10.00	11.02
22	23.5	0.51	0.53	0.51	9.20	11.00	12.12
24	25.5	0.51	0.53	0.51	10.04	12.00	13.22
27	29	0.51	0.53	0.51	11.29	13.49	14.87
30	32	0.50	0.53	0.50	12.30	14.70	16.20
33	35	0.50	0.54	0.50	13.53	16.17	17.82
36	38	0.50	0.54	0.50	14.76	17.64	19.44
42	44	0.50	0.55	0.50	17.22	20.58	22.68
48	50	0.50	0.55	0.50	19.68	23.52	25.92

Table 14 Bearing Capacity for High Strength Bolts per mm Thickness of Plate (Applicable for Grade of Bolt - 8.8, 10.9, & 12.9)

d (mm)	d_0 (mm)	$e/3d_0$	$p/3d_0 - 0.25$	k_b	Bearing Capacity, V_{dpb} (kN) per mm Thickness of Plate of Grade		
					E250	E350	E410
16	18	0.50	0.49	0.49	6.43	7.68	8.47
20	22	0.50	0.51	0.50	8.20	9.80	10.80
24	26	0.50	0.52	0.50	9.84	11.76	12.96
30	33	0.51	0.51	0.51	12.55	14.99	16.52
36	39	0.50	0.52	0.50	14.76	17.64	19.44
42	45	0.50	0.53	0.50	17.22	20.58	22.68

3.1.8.4 Bolts subjected to combined tension and shear (Refer Clause 512.5.7 of IRC:24) the bolts subjected to combined tension and shear shall be designed as per Clause 512.5.7 of IRC:24.

3.1.9 Strength of Friction Grip Bolts

In Friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts nuts shall conform to IS: 3757. Their installation procedure shall conform to IS: 4000.

3.1.9.1 Design of HSFG bolts

The design of HSFG Bolts should be done as per Clause 512.6 of IRC:24.

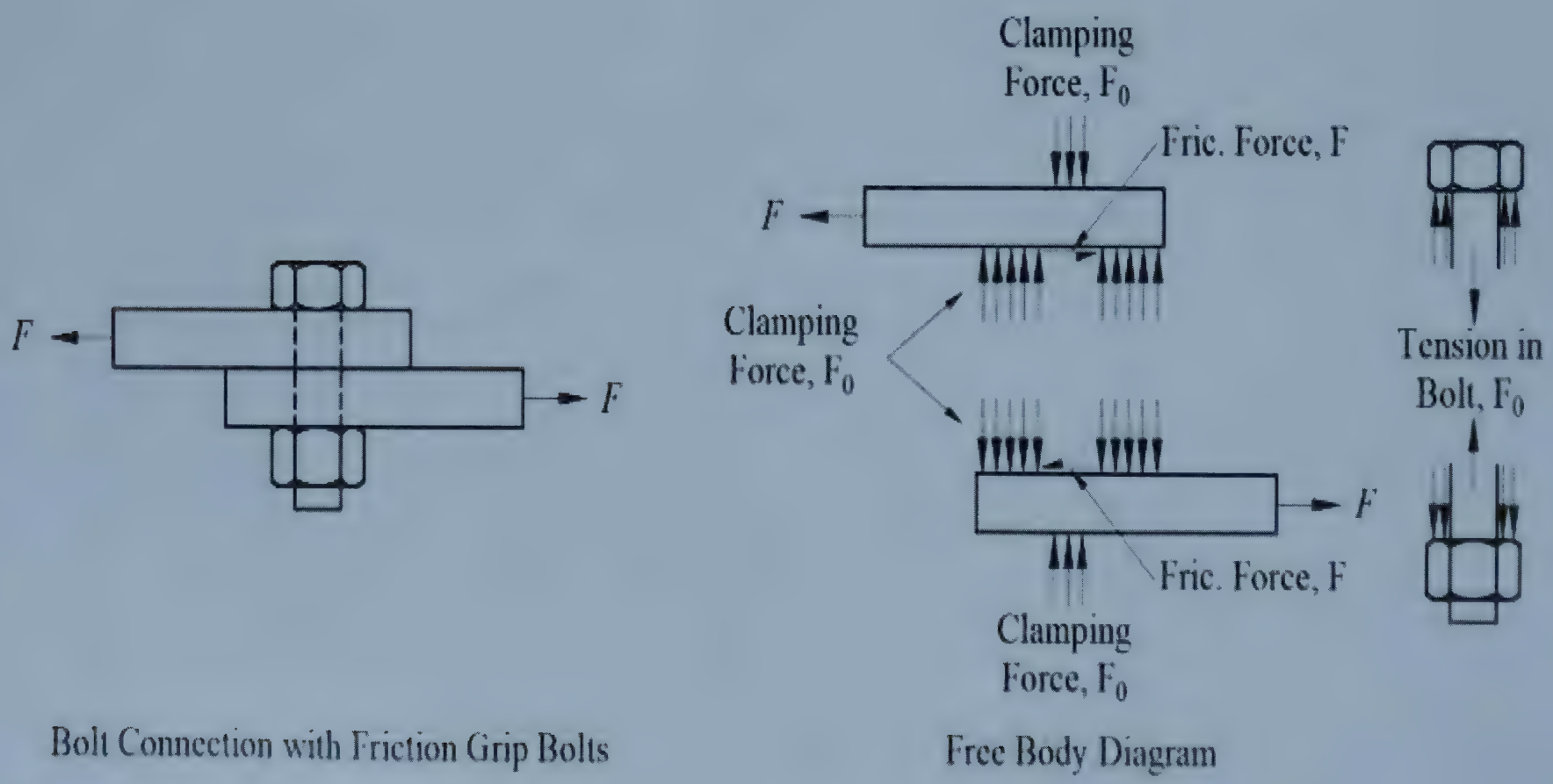


Fig. 8 Bolted Connection with HSFG Bolts

The typical average values for Co-efficient of friction may be taken from **Table 15**

Table 15 Typical Average Values for Co-efficient of Friction (μf)

Sl. No.	Treatment of Surface	Coefficient of Friction (μf)
1	Surfaces not treated	0.20
2	Surfaces blasted with short or grit with any loose rust removed, no pitting	0.50
3	Surfaces blasted with shot or grit and hot dip galvanized.	0.10
4	Surfaces blasted with shot or grit and spray-metallized with zinc (thickness 50 to 70 µm)	0.25
5	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness30 to 60 µm)	0.30
6	Sand blasted surface, after light rusting	0.52
7	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness – 60 to 80 µm)	0.30
8	Surfaces blasted with shot or grit and painted with alkali zinc silicate coat (thickness – 60 to 80 µm)	0.30
9	Surfaces blasted with shot or grit and spray-metallized with aluminium (thickness > 50 µm)	0.50
10	Clean mill scale	0.33
11	Sand blasted surface	0.48
12	Red lead painted surface	0.1

3.1.9.2 Tensioning of HSFG Bolts

High Strength Friction Grip bolts are to be tightened to capacity as per their corresponding net area & proof load which is 0.7 × $tensile\ strength$, and the same load is distributed through hardened washer to transfer the clamping load to the connecting plate.

The clamping force (A_n × 0.7 × f_{ub}) ensure that no slip will occur between the surface of the plates due to friction.

Hardened washer for HSFG Bolt to placed should conform to IS: 6649.

The direct tension indicator (DTI) washers are of special type washer which is the simplest way of ensuring that the bolting assembling are tensioned properly. DTI washers are precision engineered mechanical load cell which is reliable method of checking of HSFG Bolt tensioning.

i) **Part Turns Method of Tensioning**
Tensioning of bolts and nuts by the part turn method shall be in accordance with the following procedure:

- On assembly, all bolts and nuts in the joint shall be tightened to a snug tight condition to ensure that the load transmitting piles are brought into effective contact.
- Now turn by nut specific amount by either rotating the nut ½ turn or ¾ turns as in table 4 of IS: 4000.

ii) **Tensioning by the Use of Direct - Tension Indication**
Tensioning of bolts and nuts using a direct tension indication device shall be on, assembly all bolts and nuts in the joint shall be first tightened to a snag tight condition. After completing this preliminary tightening to snug tight the bolt and nut shall be tensioned to provide the minimum bolt tension. This shall be indicated by the tension indication device.

3.1.9.3 Sequence of Tightening of Bolt in a Joint

The bolts in a joint are not tightening simultaneously. The other preloaded bolts in the group which are already tightened has an effect when a bolt in the group is tightened. Such effect is called bolt cross talk.

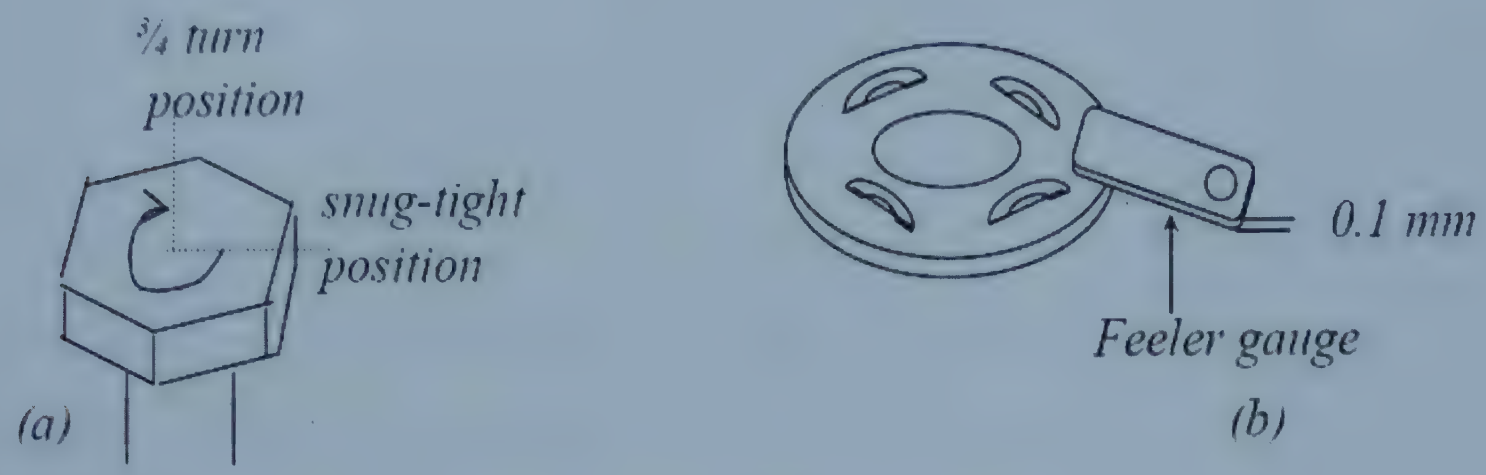


Fig. 9 Tightening of Bolt

3.1.9.4 Tightening of Bolt

$$Torque = Axial\ tension \times Nominal\ Diameter\ of\ Bolt \times Torque\ Coefficient$$

As per Clause 3.2 of IS: 4000,

$$Torque = Proof\ Load \times Nominal\ Diameter\ of\ Bolt \times Torque\ Coefficient$$

Torque Co-efficient = 0.15 if the bolt is slightly lubricated
= 0.25 if it is rusty

Some value of Torque provided for commonly used HSFG Bolts based on the proof load.

Table 16 Required Torque Values for Tightening HSFG Bolts

Bolt Diameter mm	Required Torque (N-m)			
	Slightly lubricated Bolts		Rusty Bolts	
	8.8	10.9	8.8	10.9
16	210.6	263.3	351.0	438.8
20	411.3	514.1	685.4	856.8
24	710.6	888.3	1184.4	1480.5
30	1412.7	1765.9	2354.5	2943.2
36	2469.7	3087.1	4116.2	5145.2
42	3954.5	4943.2	6590.9	8238.6

**The Above table is only a guideline. Please check with supplier at the time of execution.*

3.1.10 Forces (Refer Clause 512.7 of IRC:24)

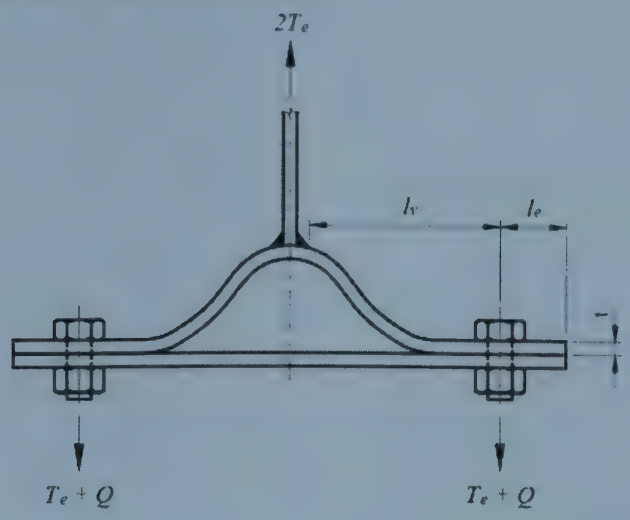


Fig. 10 Combined Prying and Tension Forces

For example,

Suppose M24 bolts (non-pre-tensioned) of Grade 8.8 is provided in a stub connection of steel grade of F_e 410 (fy = 250 MPa) with end distance = 39 mm, width of end plate = 250 mm, Thickness of stem of the stub = 20 mm, thickness of fillet weld = 8 mm, & pitch of bolt in other direction = 60 mm

$T_e = 100\ kN.$

From the provided data,

$$\beta = 2\ for\ non-pretensioned\ bolt$$

$$l_v = {}^{250}/_{2} - {}^{20}/_{2} - 8 - 39 = 68\ mm$$

$$l_e = 1.1 \times 25\sqrt{2} \times {}^{560}/_{250} = 58.2\ mm$$

Value of l_e should not be more than end distance.

Hence, $l_e = 39\ mm$

Proof Stress, $f_0 = 0.7 \times 800 = 560\ MPa$

$b_e = pitch\ distance = 60\ mm$

$\eta = 1.5$

$t = thickness\ of\ end\ plate = 25\ mm$

$T_e = 100\ kN = 100000\ N$

$$Q = \frac{68}{(2 \times 39)}\left(100000 - \frac{2 \times 1.5 \times 560 \times 60 \times 25^4}{27 \times 39 \times 68^2}\right) = 80130\ N = 80.13\ kN$$

Total Tension in bolt including Prying Force = $T_e + Q$ = 100 + 80.13 = 180.13 kN

Capacity of M24 bolts (Grade 8.8) in tension = 203 kN

Prying Force can be reduced by increasing l_e, reducing l_v & increasing end plate's thickness, t.

3.1.11 Failure of Bolt

The following are the major reasons for failure of the bolts:

i) **Fatigue:** Failure due to fatigue may occur when the stress due to cyclic load is much lower than the elastic limit. Fatigue failure can be minimized by minimizing stress level (including stress concentration), reducing load excursions, & eliminating the surface flaws of the bolts.

ii) **Strength:** The tensile strength of the bolt under static load will depend on the terms ultimate, yield, or proof strength. If the applied force is more than the ultimate tensile capacity, bolt will fail. When bolt load exceeds its shear & bearing capacity, the bolt will automatically fail. Number of bolts or diameter of the bolt to be increased.

iii) **Stripping of Thread:** Strength of thread depends on the load carrying capacity of the bolt through its engagement with the thread of the nut. If the strength of the thread is lower than the applied load, then stripping of thread will occur. Stripping areas for internal threads should be more than that of external threads.

iv) **Corrosion:** The essential condition for corrosion are cathode, anode, electrolyte and conductor. When the bolt will be stressed, it will act as anode and a tiny crack will develop on the surface. Crack which is in turn will increase thus reduces the area of the bolt, cause failure due to corrosion. The most common way to reduce corrosion problem is to make the bolts of corrosion resistant material or by the way of protecting the bolt by providing coat of protective layer.

v) **Others:** Bolt will also fail due to the quality of the materials, temperature (differential expansions/contractions) and self-loosening. Self-loosening can be prevented by locking the nut to the bolt by mechanical or chemical means.

3.2 Welds

Welding is the process of joining steel members to create the desired component for a steel bridge structure.

3.2.1 Type of Welds

There are two main types of welds which are used in producing components for a steel bridge:

i) Butt Welds
ii) Fillet Welds

3.2.1.1 Butt Weld (or Groove Weld)

Butt weld is defined as one in which the metal lies substantially within the planes of the surfaces of the joined components.

Complete Joint Penetration (CJP): It is a full penetration butt weld and is normally expected to develop full strength.

Partial Joint Penetration (PJP): A partial penetration weld achieves a specified depth of penetration only. These are to be, preferably, used in a restrictive manner in bridge applications.

Groove welds can be subdivided into square, bevel, V, J and U types depending on the shape to which the plate edge is prepared or grooved to accommodate the weld.

The typical angles for different Groove joint preparations are:

- 25-30 degree for a V preparation

- 8-12 degree for a U preparation
- 40-50 degree for a single bevel preparation
- 10-20 degree for a J preparation

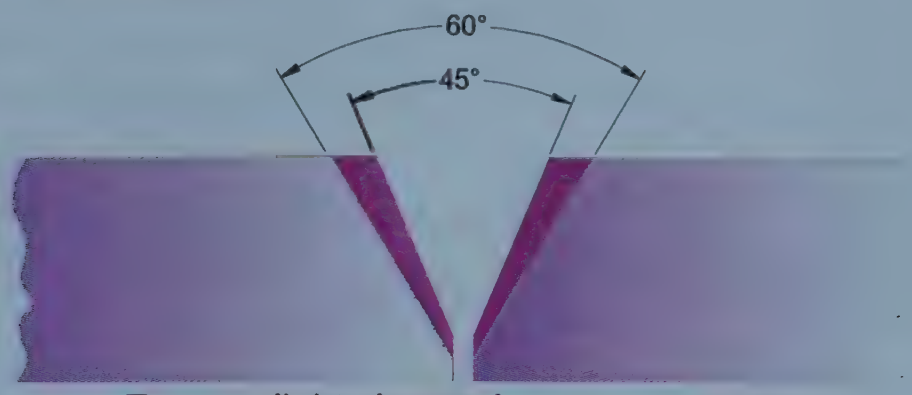


Fig. 11 Typical Groove Joint

The angle and groove type depends on the type of structure design, welding processes, and consideration of welding cost. The area of the metal's surface that is melted during the welding process is called the faying surface. The faying surface can be shaped before welding to increase the weld's strength. Shaping of the weld joint is called edge preparation. The edge preparation may be the same on both members of the joint, or each side can be shaped differently. Points to be considered at the time of Edge preparations are:

a) **Root Face:** The portion of a fusion face at the root that is not beveled or grooved. Its value depends on the welding process used, parent material to be welded and application. For a full penetration weld on carbon steel plates, it has a value of $\mathbf{1 - 2}\ \boldsymbol{mm}$ (for the common welding processes).

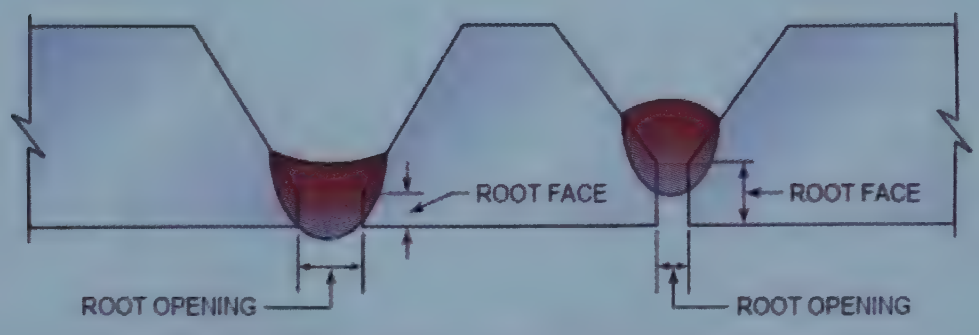


Fig. 12 Root Face

b) **Root Gap (Root Opening):** Minimum distance at any cross-section between edges ends or surfaces to be joined. Its value depends on the welding process used and application. For a full penetration weld on carbon steel plates, it has a value between $\mathbf{1 - 4}\ \boldsymbol{mm}$.

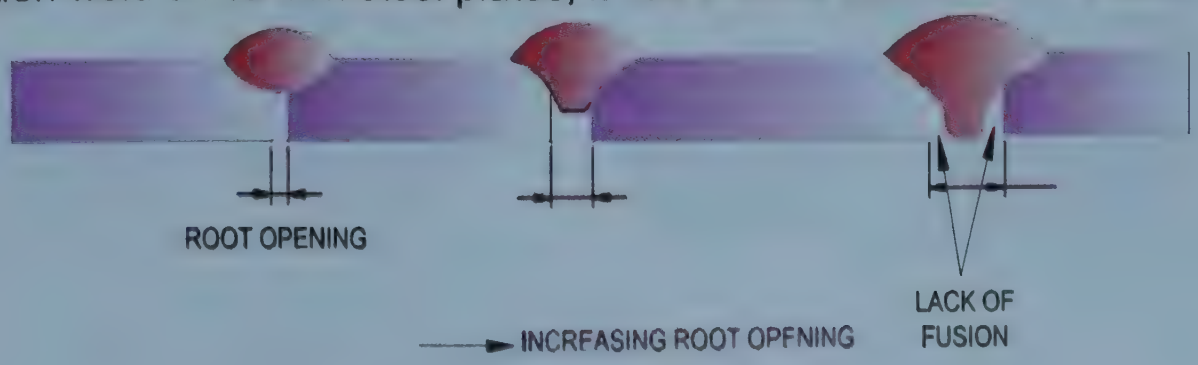


Fig. 13 Root Gap or Root Opening

c) **Root Radius:** The radius of the curved portion of the fusion face in a component prepared for a single J, single U, double J or double U weld. In case of carbon steel plates, the root radius has a value of 6 mm for single and double U preparations and 8 mm for single and double J preparations. Typical Preparations are illustrated below.

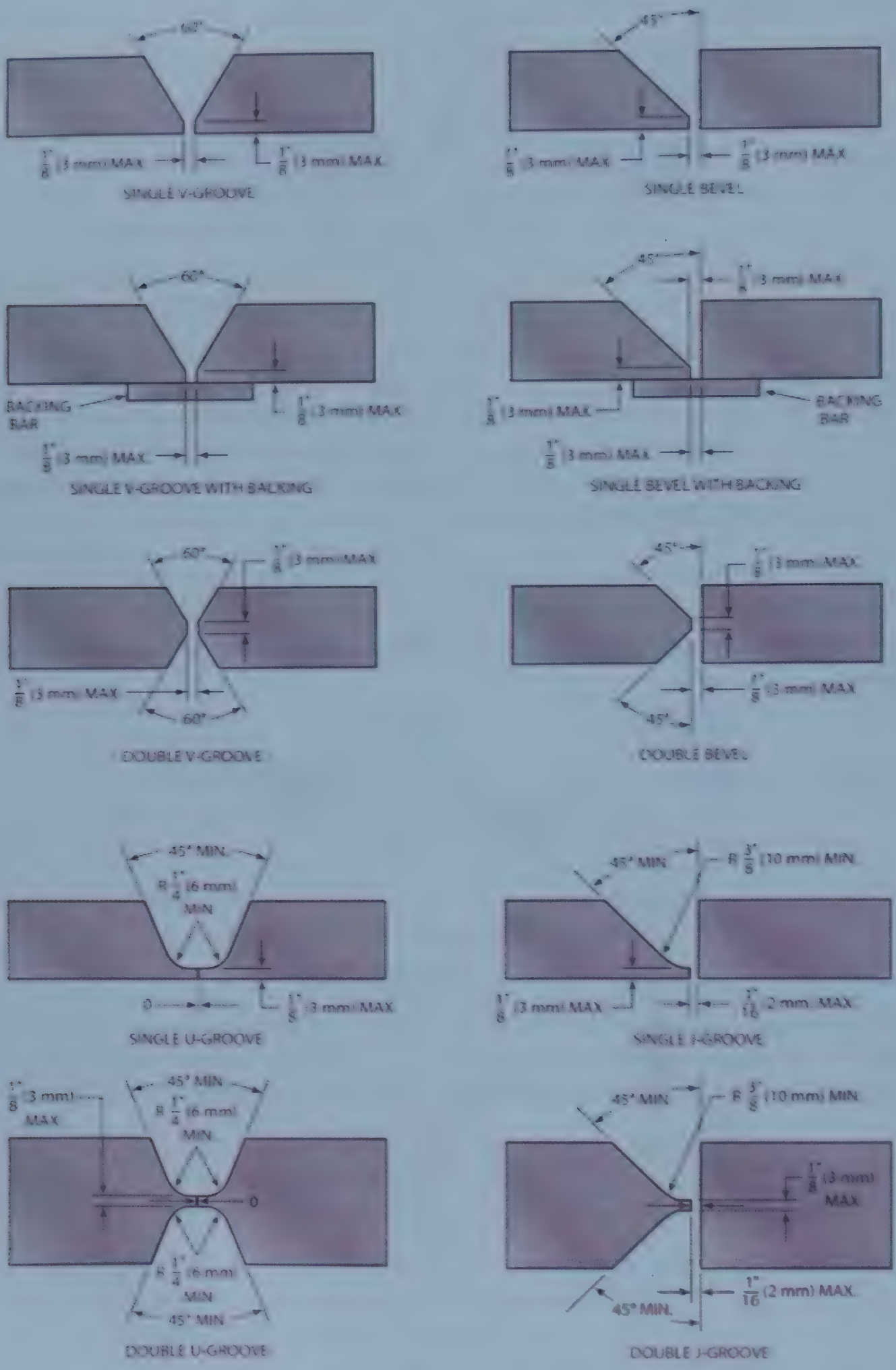


Fig. 14 Root Radius

For additional information on butt weld, IS: 9595 may be referred to.

3.2.1.2 Fillet Weld

A fillet weld is approximately triangular in section, formed within a re-entrant corner of a joint. The weld size is specified as the leg length. The strength is achieved through shear capacity of the weld metal across the throat.

The most commonly used welding processes are as follows:

i) SMAW / MMA – Shielded Metal Arc Welding / Manual Metal Arc Welding
ii) GMAW – Gas Metal Arc Welding
iii) SAW – Submerged Arc Welding
iv) FCAW – Flux Cored Arc Welding

Welding Positions: The term 'position' when used in connection with a welded joint refers to the position in which the bead is laid.

i) Flat – 1G, 1F
ii) Horizonal – 2G, 2F
iii) Vertical – 3G, 3F
iv) Overhead – 4G, 4F

The welding position is of great importance since the quality of the weld is directly affected by the manner in which the weld metal is deposited. Not all electrodes are suitable for all positions; many electrodes are specified for use in a particular position and should be used only in that position to obtain good results.

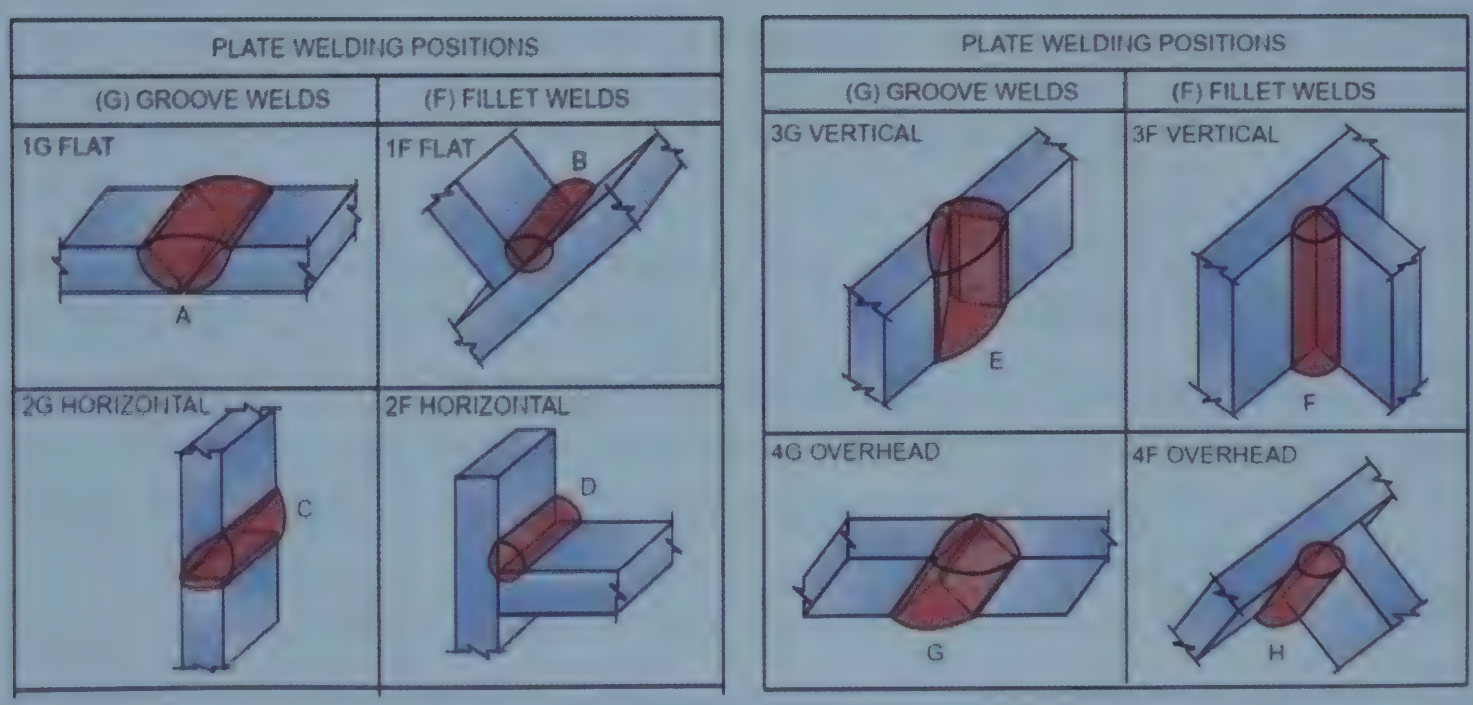


Fig. 15 Plate Welding Positions

3.2.2 Weld Symbol

Becoming a professional welder is rewarding and provides a lifelong opportunity to enjoy fulfilling employment worldwide. There are many challenges along the way, and most are practical skill-based challenges. However, sooner or later one will be faced with drawings that one is required to weld from. It is helpful to understand welding symbols so that when the concerned engineer is faced with complicated drawings, the symbol is one less obstacle to overcome.

Some welding symbols look complicated, but when they are broken down, it will be observed that they are quite simple. The symbols are illustrations of the pre-weld joint looking side on, as through a cross- section. Each symbol is explained individually, with its weld profile alongside it.

There are two systems that are used for interpretation, and four sections of different symbols are explained in this standard – the base platform, base butt weld symbols, other base symbols, and supplementary symbols.

3.2.2.1 Two Drawing Systems

There are two systems the base symbol comes in, and they are each interpreted differently. The specific system will be indicated on the plans, and both systems will not be included in one drawing.

Base System A is distinguished by the dash lines underneath the reference line. As the image illustrates, when the welding symbol is on the dashed line side, the weld needs to be on the opposite side of the arrow. When the welding symbol is on top of the reference line, the weld needs to be on the same side as the arrow. Sometimes drawings are very full, so this rule is necessary in case there is no room for the base symbol to be on the side the weld is.

Base System B is recognized by the absence of dashes below the reference line. Here, when the welding symbol is on the underside of the reference line, the weld is on the side the arrow points at. When the symbol is on the top side of the reference line, the weld is on the opposite side of the pointer.

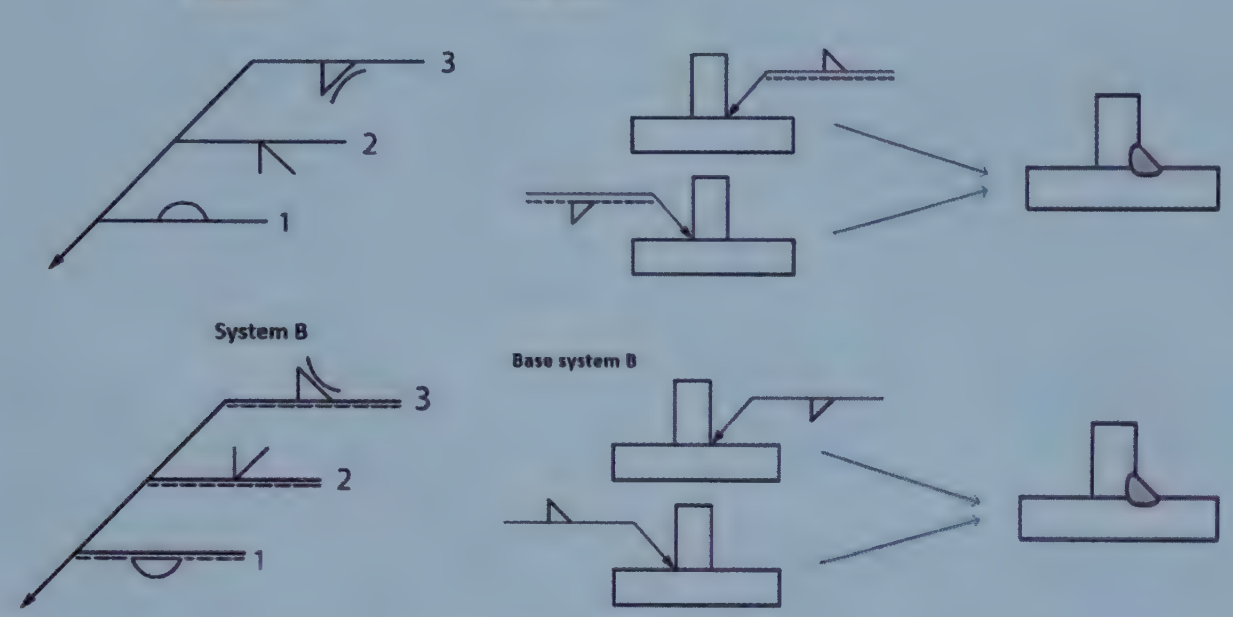


Fig. 16 Base Systems for Interpretation of Weld

3.2.2.2 Different Symbols

a) **Base Platform**: This symbol is a simple platform for displaying the characteristics and surrounding details for welds.

It has three parts:

i) The Arrow Line: This points to the general location of the weld.
ii) The Reference Line: This is where the details are placed about the type of weld and the specific location.
iii) The Tail: This is where the complementary details, separate from the specifics, are placed. This includes welding standards, material types, and the welding process required.

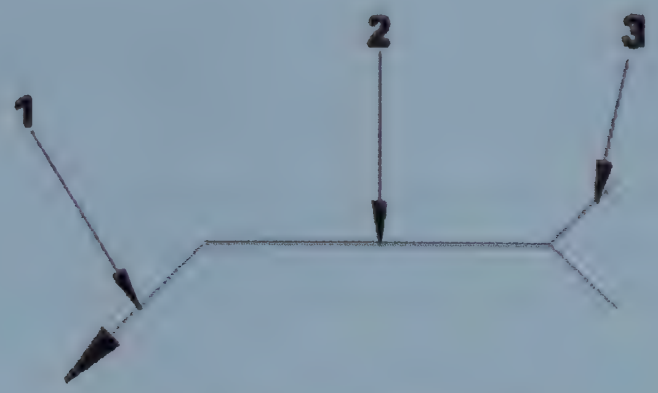


Fig. 17 Base Platform

b) **Base Butt Symbols**: As the name suggests, these joints are distinguished by their butting up to each other end to end. For example, two plates sit flat on their faces aligned parallel and straight across their tops, and they are welded on one or both sides depending on the symbol.

i) **Single Sided Butt Weld:** These joints are only welded on one side and are easily recognized as single-sided by their symbol. The symbols reveal their required weld prep, and the type of weld needed is determined by other symbols.

ii) **Square Butt Weld:** This joint has no weld prep. Both plates have square ends, just as the symbol displays two square angles.

iii) **Single V Butt Weld:** The single V butt joint has a 45° weld prep on one side of each plate the full depth of the material. The joined plates will form a V shape as the symbol indicates.

iv) **Single V Butt Weld with Broad Root Face:** Similar to the single V butt, this weld prep has a 45° angle on one side of each plate from the top of the metal to no more than ¾ of the way down the plate depth, leaving a section of material below. The joint imitates its symbol as a Y.

v) **Single Bevel Butt Weld:** This butt prep has a 45° angle cut the full depth on one side of a single plate, while the other end is square as the symbol displays.

vi) **Single Bevel Butt Weld with Broad Root Face:** With one square end and a 45° bevel on the other side only partly through the plate, the weld prep will look like its symbol, as "r" without an arc.

vii) **Single U Butt Weld:** Both plates have a corner cut out in a ¼ moon shape about ¾ of the plate gauge deep to form a U partway through the joined sections, as the symbol shows.

viii) **Single J Butt Weld:** One plate has a square end, while the other has a corner cut out in a ¼ moon shape partway through the plate thickness, forming a mild J when joined, as you can see in the symbol.

ix) **Double Sided Butt Joints:** These welds are precisely the same as single-sided butt joints, except both sides are prepped and welded.

x) **Double Sided V Butt Joint:** Both ends of each plate have a 45° bevel cut out meeting halfway through the plate to form a point. When joined together, the points meet. This creates an X as seen in the symbol, the full depth of the joint.

xi) **Double Sided Bevel Butt Joint:** One plate has no weld prep, keeping its corners square. The other side has two 45° bevels finishing along the middle to make a point. As the symbol displays, the joined plates will form a K.

xii) **Double Sided U Butt Joint:** Both sides of the two plates have a ¼ moon cut out, leaving a section about one-quarter to one-half the plate gauge in the center between the two weld preps. When joined together, they make a U on top of an n, just as you see in the symbol.

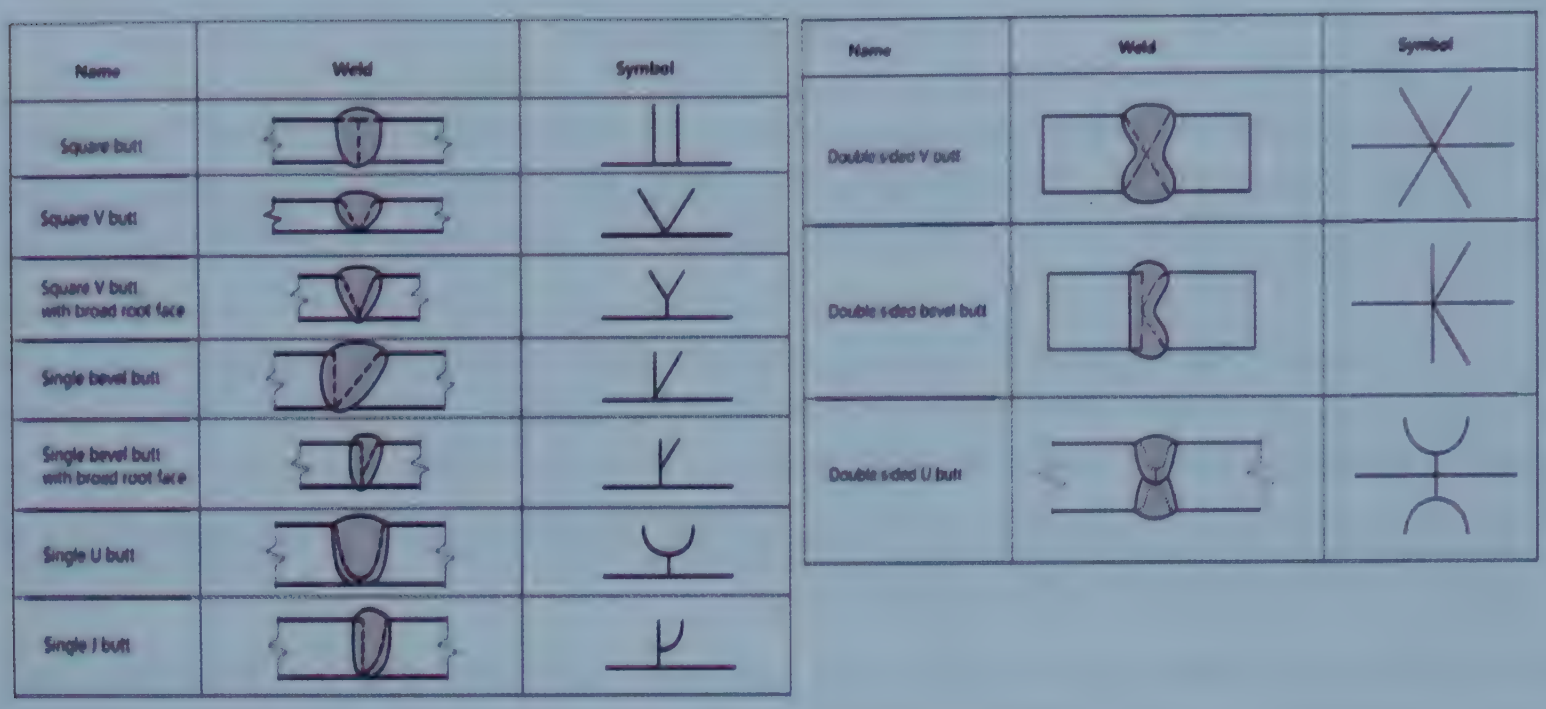


Fig. 18 Base Butt Symbols

c) **Other Base Symbols**: Butt welds and fillet welds are the most common types of stitch. However, unlike the majority of butt welds, the different kinds of fillet welds are indicated by supplementary symbols added to the fillet symbol. Following are all the other base welding symbols which are not butt welds, including the fillet symbol.

i) **Fillet:** This symbol has a right-angled triangle, since the most common fillet joint is between two plates at a right angle to each other. Usually, it's between a 90° joint, or somewhere close.

ii) **Plug:** One plate with a hole sits on top of another plain plate. The plug weld fuses the two plates together, with a weld inside it completely filling the hole. Its symbol is the least clear of them all, but it displays the plug's hole in a cross-section view of the top plate.

iii) **Resistance Spot:** Two plates are fused together by creating heat from electrical resistance between the plates with a resistance spot welder. Its symbol is a circle, the same shape as a spot weld.

iv) **Resistance Seam:** This is like a resistance spot weld, only the electrical resistance creates a long seam weld rather than a single spot. Similar to the spot welds symbol, the seam weld has two parallel lines through the circle symbolizing the seam weld having length like a slot shape. The resistance seam creates a circle shape which is spread out lengthwise.

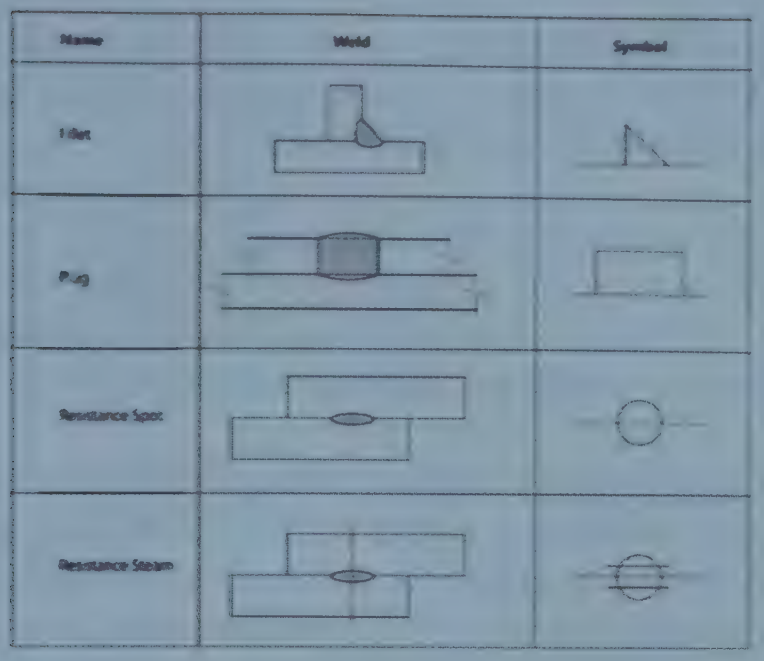


Fig. 19 Other Base Symbols

d) **Supplementary Symbols:** These symbols are added to the base symbols to clarify the type of weld required. They include characteristics of the weld, how and where to weld it, and the finishing features needed.

i) **Flush Finish:** This symbol signifies that the weld needs to be machined or ground back flush to the level of the rest of the plate. Its symbol is a straight line showing what the finished surface will look like.

ii) **Convex:** The convex finish of a weld curves outwards like a balloon away from the weld, just as its symbol displays.

iii) **Concave:** The concave's symbol is curved the opposite direction from the convex, symbolizing that the weld should be finished with an inward curve like the inside of a bowl. It is often a finish used with fillet welds.

iv) **Backing Weld:** This is when the bottom of a weld prep like a V or U needs a small initial weld at the weld prep's base before fully welding it. It gives substance for the full weld to take place without blowing through from excessive heat when only a thin section of material remains. Its symbol as a curved line is placed underneath or on top of the base symbol, depending on what side of the reference line the base symbol is on.

v) **Consumable Insert:** Inside a weld prep, a consumable plug is inserted at the bottom where there is space between the two sections. The insert is fused in with the weld and stops the bottom from between the two sections. The insert is fused in with the weld and stops the bottom from blowing out. Its symbol is the shape of a round insert looking at it side on.

vi) **Weld Entirely Around:** This symbol circles the corner of the base platform to show that the section being pointed at needs welding the whole way around it, like a circle.

vii) **Weld Between Points:** The drawing will show two points like an X and a Y, for example, between sections needing welding. The arrows of the symbol between the two letters indicate that a weld needs to run the full length between the two marks.

viii) **Weld on Site:** This symbol has a flag showing the weld needs to be done on site, not in the workshop. Just as a flag is positioned to mark a country's site, so this flag indicates it must be welded at the project's site.

ix) **Staggered Intermittent Weld:** This requires welds to be staggered intermittently across side one and side two of a section. Don't weld the full length of both sides. Instead, weld even stitches across side one. Then imitate the welds along side two, but in between side one's welds, not aligned with them.

x) The Z-shaped symbol with a line passing through its center displays the effect of a 7 in a mirror. It shows two 7s equally proportioned but flipped, making a Z. Likewise, the staggered intermittent weld is equal on both sides of a feature, but flipped so that the welds sit in different places but with the same spaces.

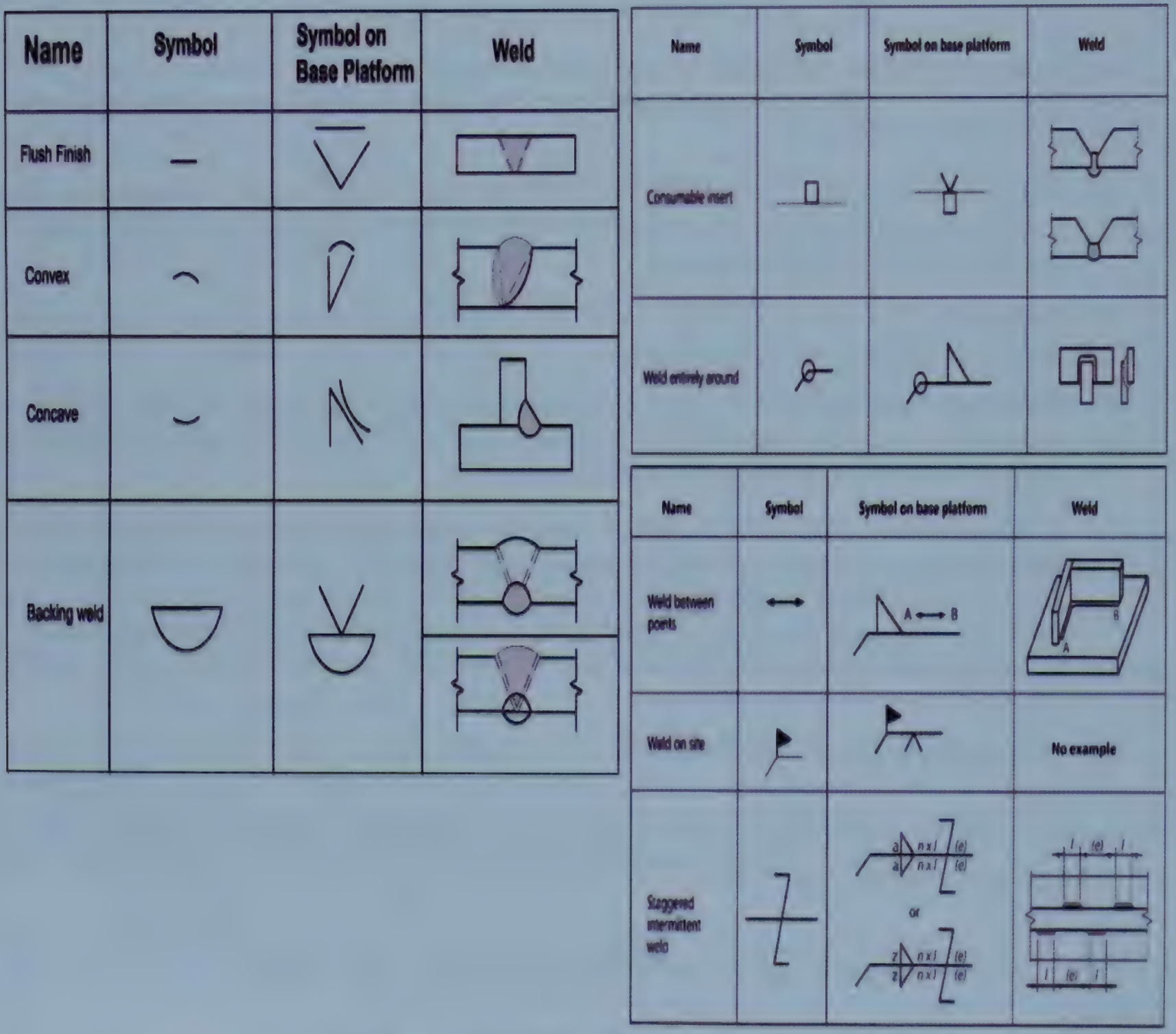
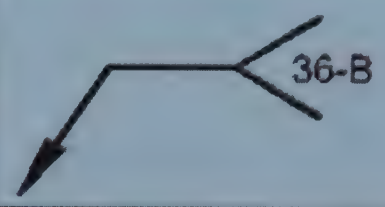
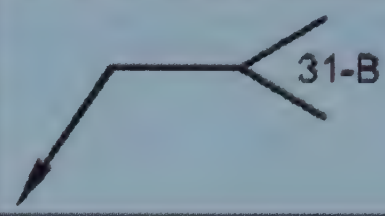
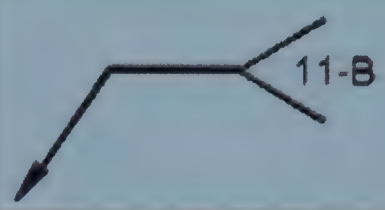
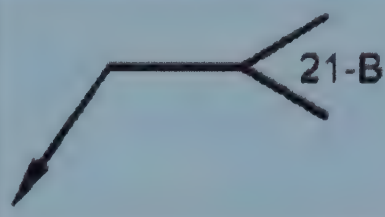
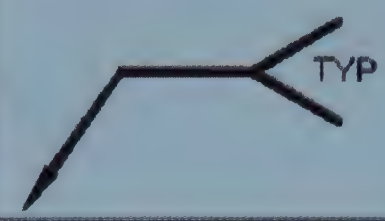


Fig. 20 Supplementary Symbols

3.2.3 Detailing Convention

Table 17 Detailing Convention

Sl. No.	Name	Nomenclature	Application/ Grade	Symbol	Characteristics of consumables
1	Flux Cored Arc Welding (FCAW)	136	Construction: both Inside & Outside/ CLASS B		Continuous consumable electrode filled with flux. Manual & Automatic Process.
2	Gas Metal Arc Welding (GMAW)	131	Construction: both Inside & Outside/ CLASS B		Continuous consumable electrode and shielding gas. Manual & Automatic Process.
3	Shielded Metal Arc Welding (SMAW)	111	Construction: Outside/ CLASS B		Consumable electrode covered in flux, can weld any metal as long as they have the right electrode. Manual Process.
4	Submerged Arc Welding (SAW)	121	Construction: Inside/ CLASS B		Arc submerged in granular flux. Automatic Process.
5	Unspecified		Construction: both Inside & Outside and any position/ CLASS B		Typ. denotes any of the welding process serial No from 1 to 4 can be adopted.

Welding Classes for Structural Application

a) **Class A** welds are intended for rigorous applications and where load bearing is primary consideration. The weld is normally radiographic quality.

b) **Class B** welds are intended for less critical applications where lower ductility, greater porosity and larger discontinuities can be tolerated.

c) **Class C** welds are intended for applications where load bearing is not a primary consideration.

3.2.4 Examples of Weld Detailing (Bow String and Plate Girders):

a) Bow String Girder

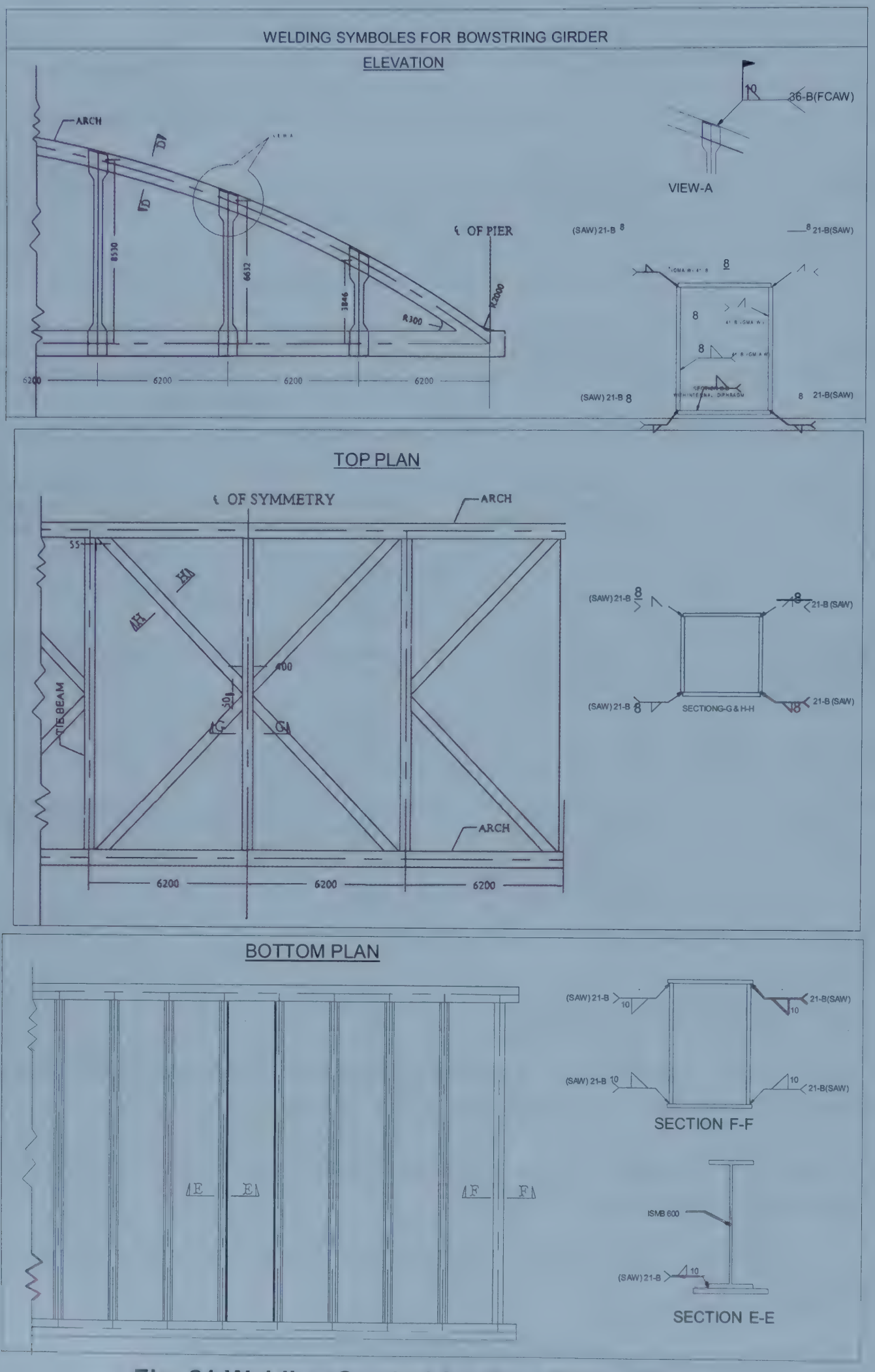
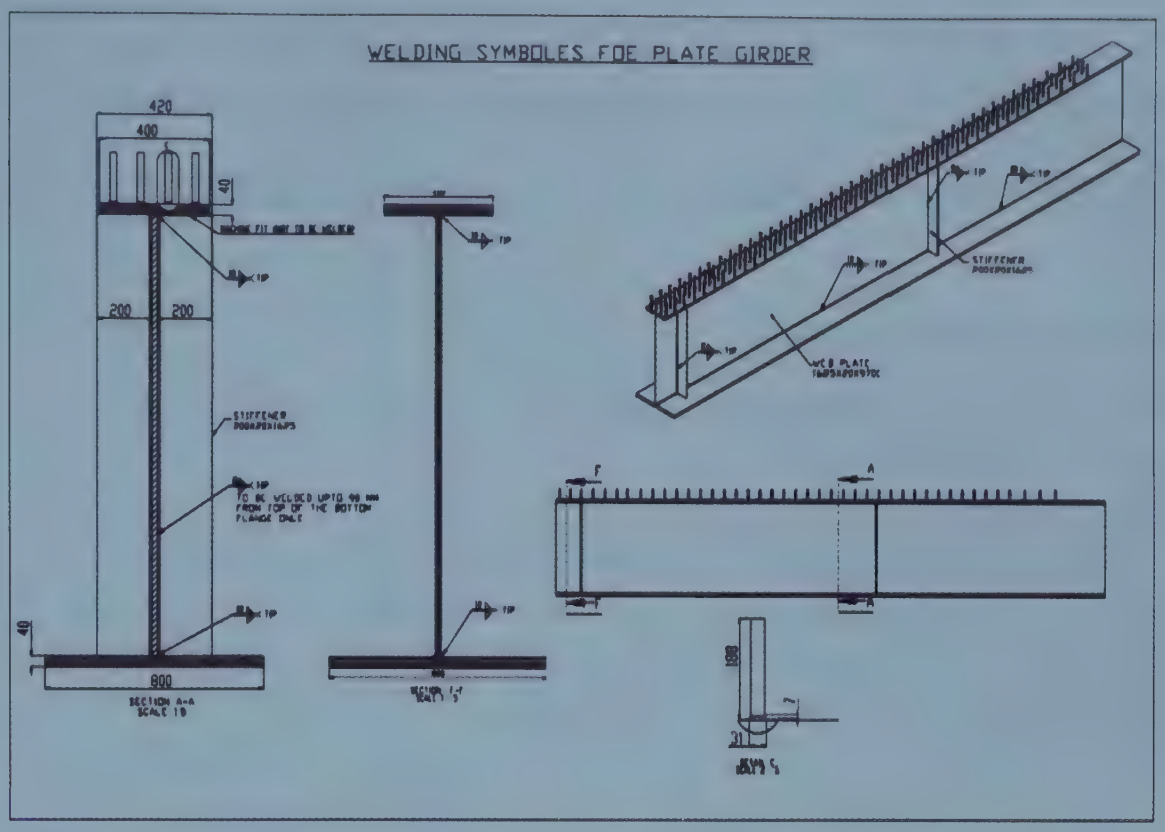
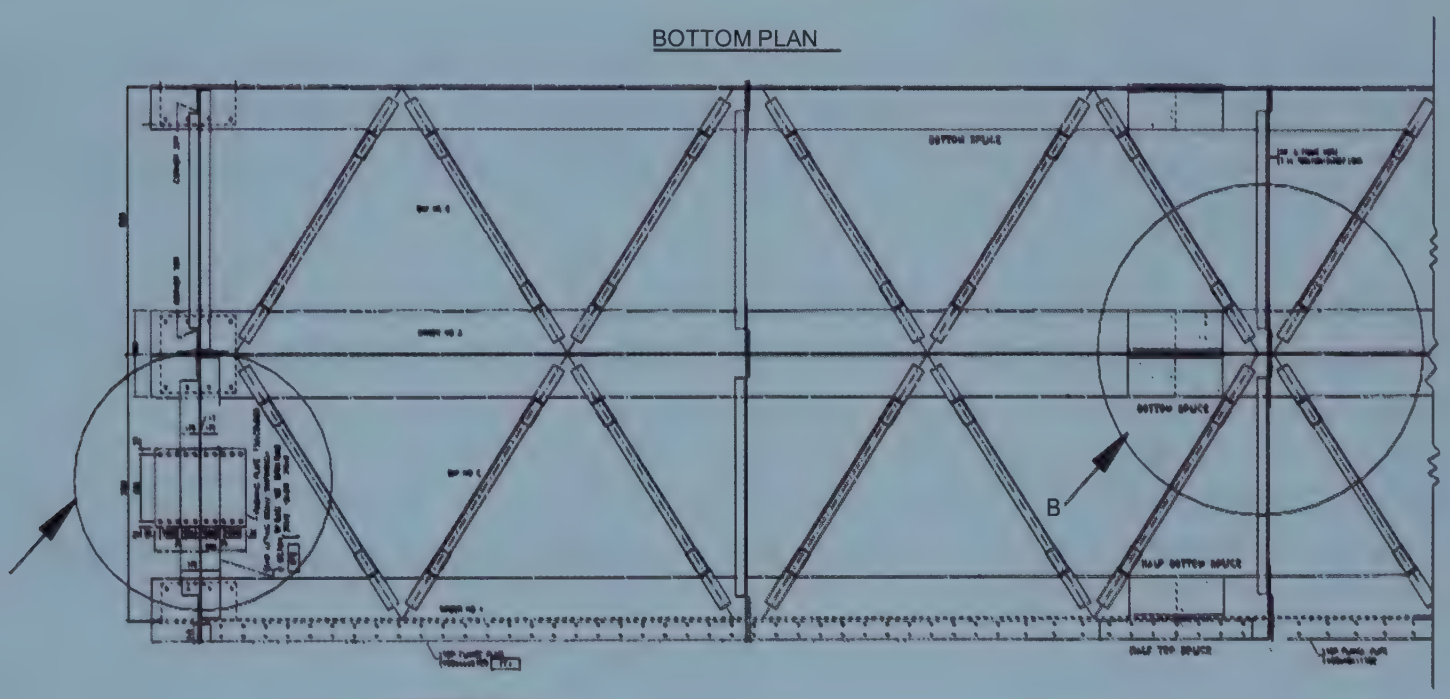
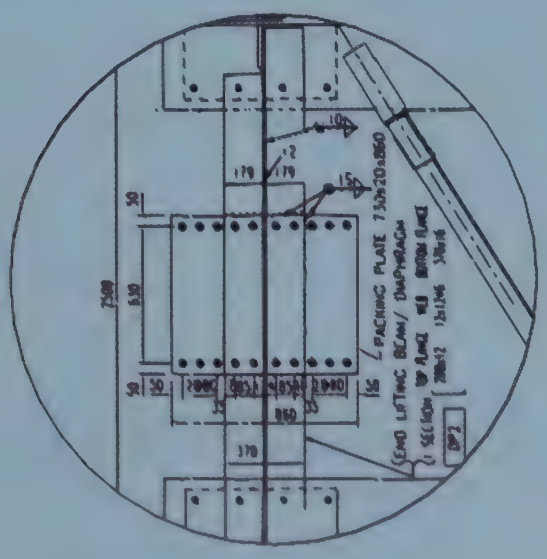


Fig. 21 Welding Symbol for Bow String Girder

b) Plate Girder







DETAIL OF "A"

DETAIL OF "B"

Fig. 22 Welding Symbol for Plate Girder

3.2.5 Design Strength-Fillet and Butt Weld

Welded joints are often crucially important affecting the safety of the design systems. It is important that the notes and data below are only used for preliminary design evaluations.

Final detail design should be completed in a formal way by using appropriate and relevant codes and standards.

Variables Related to Welded Joints:

i) Strength of deposited weld material
ii) Type of joint and weld
iii) Size of weld
iv) Location of weld in relation to parts to be joined
v) Types of stress to which the weld is subject to
vi) Conditions under which weld is carried out
vii) Type of equipment used for welding
viii) Skill of welder

A generous factor of safety should be used and if fluctuating loads are present, then additional design margins should be included to allow for fatigue. The minimum amount of filler material consistent with the job requirement is to be used. Attempt shall be made to design the joint such that load path is not through the weld.

Table 18 provides approximate value of stresses. In case of direct loading, the butt weld stresses are tensile/ compressive (σ_t). For the fillet welds the stresses are assumed to be shear r_s applied to the weld throat.

For butt welded joints subject to bending the butt weld stresses result from a tensile/compressive stress (σ_b) and a direct shear stress r_s. In this case, the design basis stress should be calculated as

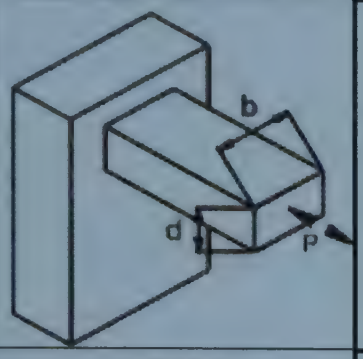
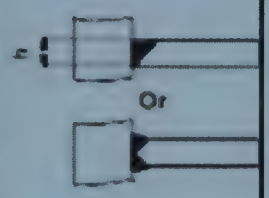
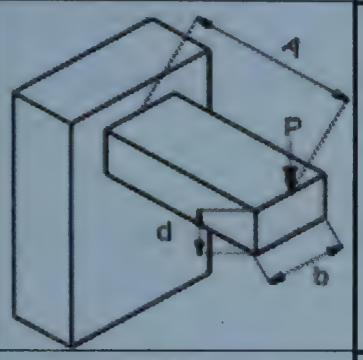
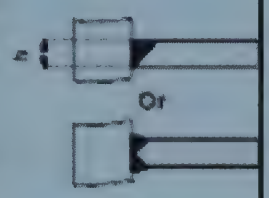
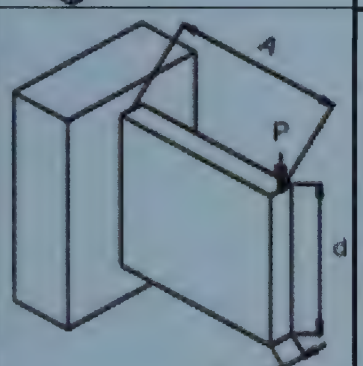
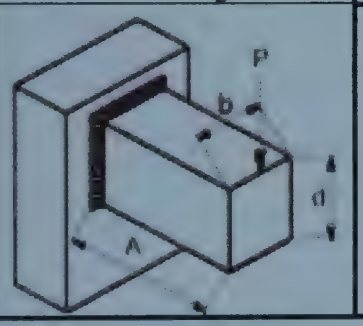
$$\sigma_r = \sqrt{(\sigma_b^2 + 4\, r_s^2)}$$

For fillet welded joints subject to bending, the stresses are all shear stresses. From bending r_b and from shear r_s, the design basis stress is given by

$$r_r = \sqrt{(r_b^2 + r_s^2)}$$

The stresses from joints subject to torsion loading include shear stress from the applied load and shear stresses from the torque loading. The resulting stresses should be added vectorially taking care to choose the location of the highest stresses.

Table 18 Bracket Weld Subject to Direct and Bending Stresses

Method of Loading	Weldment	Stress in Weld; Weld size (h)	Weldment	Stress in WeldWeld size (h)	Weldment	Stress in Weld Weld size (h)
		$\sigma_t = \frac{P}{b.h}$ $h = \frac{P}{b\,\sigma_t}$		$\sigma_t = \frac{0,5.P}{b.h}$ $h = \frac{0,5.P}{b.\sigma_t}$		$\tau_s = \frac{0,71.P}{b.h}$ $h = \frac{0,71.P}{b.\tau_s}$
		$\sigma_b = \frac{6.P.A}{b.h^2}$ $\tau_d = \frac{P}{b.h}$ $h = \sqrt{\frac{6.P.A}{b.\sigma_b}}$		$x = d^3-(d-2h)^3$ $\sigma_b = \frac{6.P.A.d}{b.x}$ $\tau_s = \frac{P}{2.b.h}$		$x = (d+2h)^3-d^3$ $\tau_p = \frac{6,5.P.A.(d+2h)}{b.x}$ $\tau_s = \frac{0,71.P}{b.h}$
		$\sigma_b = \frac{6.P.A}{h.d^2}$ $\tau_s = \frac{P}{d.h}$ $h = \frac{6.P}{\sigma_{II} d^2}$		$\sigma_b = \frac{3.P.A}{h.d^2}$ $\tau_s = \frac{P}{2.d.h}$ $h = \frac{3.P.A}{\sigma_b d^2}$		$\tau_h = \frac{4,24.P.A}{hd^2}$ $\tau_s = \frac{0,71.P}{hd}$ $h = \frac{2,12^2.P.A}{\tau_p d^2}$
				$x = (b-2h).(d-2h)^3$ $y = (b-2h).(d-2h)$ $\sigma_b = \frac{6.P.A.d}{bd^3.x}$ $\tau_s = \frac{P}{(bd.y)}$		$x=(b+2h).(d+2h)^3$ $y=(b+2h).(d+2h)$ $\tau_p = \frac{8.49.P.A.d}{x-b.d^3}$ $\tau_s = \frac{1,41.P}{(y.bd)}$

3.2.6 Assessment of Fillet Weld Groups

The method of assessing fillet welds groups treating welds as lines is reasonably safe and conservative and is very convenient to use. The method is based on the simple method of calculation of weld stress.

The other method, the direction method, uses the method of resolving the forces transmitted by unit thickness welds per unit length into traverse forces (F_T) and longitudinal forces (F_L).

a) **Weld Subject to Bending:** A fillet weld subject to bending is easily assessed as follows:

The area of the fillet weld (A) is calculated assuming unit thickness.

The moment of inertia (I) is calculated assuming unit thickness.

The maximum shear stress due to bending is determined as follows:

$$r_b = M \times y/I$$

The maximum shear stress due to direct shear is determined as follows:

$$r_s = P/A$$

The resultant stress is calculated as follows:

$$r_r = \sqrt{(r_b^2 + r_s^2)}$$

By comparing the design strength ρw with the resultant stress r_r, the value of the weld throat thickness is calculated and then the weld size i.e. if $r_r/\rho_w = 5$, then the throat thickness t = 5 units and the weld leg size h = 1.414t.

b) **Weld Subject to Torsion:** A fillet weld subject to torsion is easily assessed as follows:

The area of the fillet weld (A) is calculated assuming unit thickness.

The polar moment of inertia (J) is calculated assuming unit thickness.

$$J = I_{xx} + I_{yy}$$

The maximum shear stress due to bending is determined as follows:

$$r_t = T \times r/J$$

The maximum shear stress due to direct shear is determined as follows:

$$r_s = P/A$$

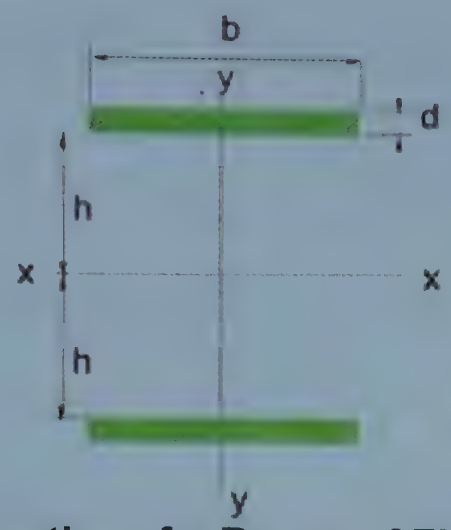
The resultant stress (r_r) is the vector sum of r_t and r_s

By comparing the design strength ρw with the resultant stress rr, the value of the weld throat thickness is calculated and then the weld size i.e., if $r_r/\rho_w = 5$, then the throat thickness t = 5 units and the weld leg size h = 1.414t.

3.2.7 Properties of Weld Groups with Welds Treated as Lines:

It is accepted that it is reasonably accurate to use properties based on unit weld thickness in calculation to determine the strength of welds.

The weld properties I_{xx}, I_{yy} and J are assumed to be proportional to the weld thickness. The typical accuracy of this method of calculation is shown below:



$I_{xx} = 2\,[(bd^3/12) + bd(h+0.5d)^2]$

If weld thickness d is small compared to h and b then

$I_{xx} = 2\,bdh^2$ with little loss of accuracy

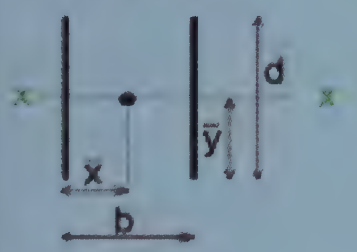
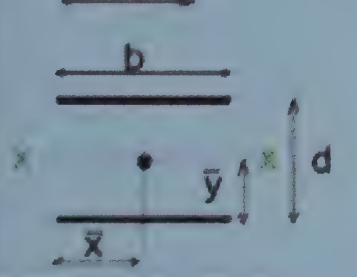
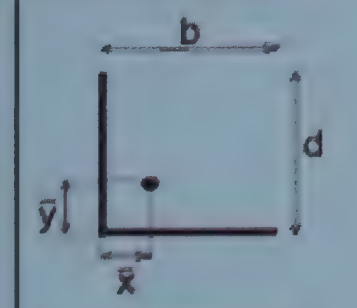
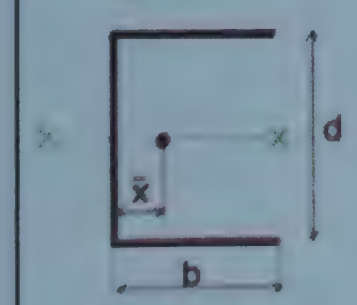
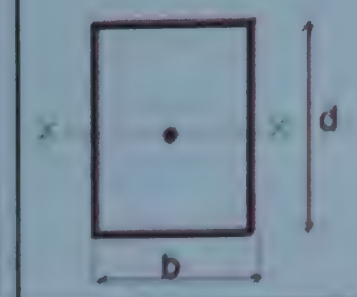
$I_{yy} = 2\,(db^3/12)$

$J = I_{xx} + I_{yy}$

It is clear when d is small compared to b and h

then J , I_{xx} I_{yy} are all proportional to the width of the weld (d)

Properties of a Range of Fillet Weld Groups with Welds Treated as Lines:

Weld	Throat Area Unit Area	Location of COG x, y	I_{xx}	J
	0,707hd d	$\bar{x} = 0$ $\bar{y} = d/2$	$I_{xx} = \frac{d^3}{12}$	$J = \frac{d^3}{12}$
	1,414hd 2d	$\bar{x} = b/2$ $\bar{y} = d/2$	$I_{xx} = \frac{d^3}{6}$	$J = \frac{d\,(3b^2 + d^2)}{6}$
	1,414hb 2.b	$\bar{x} = b/2$ $\bar{y} = d/2$	$I_{xx} = \frac{bd^2}{2}$	$J = \frac{b\,(3d^2 + b^2)}{6}$
	0,707 h (b+d) b + d	$\bar{x} = \frac{b^2}{2(b+d)}$ $\bar{y} = \frac{d^2}{2(b+d)}$	-	$J = \frac{(b+d)^4 - 6b^2d^2}{12(b+d)}$
	0,707h(2b+d) 2b +d	$\bar{x} = b^2/(2b+d)$ $\bar{y} = d/2$	$I_{xx} = \frac{d^2(6b+d)}{12}$	$z = 8b^3 + 6bd^2 + d^3$ $J = \frac{z}{12} - \frac{b^4}{2b+d}$
	1,414 h(b+d) 2(b +d)	$\bar{x} = b/2$ $\bar{y} = d/2$	$I_{xx} = \frac{d^2(3b+d)}{6}$	$J = \frac{(b+d)^3}{6}$

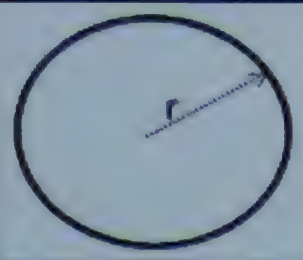
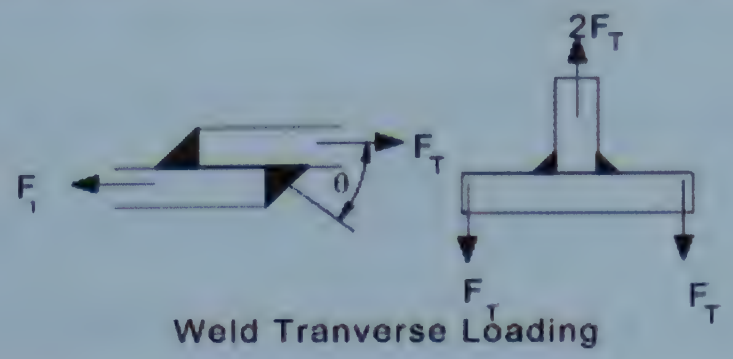
	1,414 πhr 2πr	-	$I = \pi r^3$	$J = 2\pi r^3$

Table Of Weld Capacities

The fillet weld capacity tables related to the type of loading on the weld.

Two types of loading are identified traverse loading and longitudinal loading as show below:



Weld Tranverse Loading

Weld Longitundinal Loading

The weld loading should be such that

$$\{(F_L/P_L)2 + (F_T/P_T)2 \leq 1$$

Design Strength of P_L and P_T.

$$P_L = a \times \rho w$$

$$P_T = a \times K \times \rho w$$

a = weld throat size.

$$K = 1.25 \times \sqrt{(1.5/1 + Cos^2\theta)}$$

P_T is based on elements transmitting forces at 90° i.e., $\theta = 45°$ and $K = 1.25$

Weld Capacity E35 Electrode S275 Steel				Weld Capacity E42 Electrode S355 Steel			
Leg Length	Throat Thickness	Longitudinal Capacity	Transverse Capacity	Leg Length	Throat Thickness	Longitudinal Capacity	Transverse Capacity
		P_L(kN/mm)	P_T (kN/mm)			P_L	P_T
mm	mm	kN/mm	kN/mm	mm	mm	kN/mm	kN/mm
3	2,1	0,462	0,577	3	2,1	0,525	0,656
4	2,8	0,616	0,720	4	2,8	0,700	0,875
5	3,5	0,770	0,963	5	3,5	0,875	1,094
6	4,2	0,924	1,155	6	4,2	1,050	1,312
8	5,6	1,232	1,540	8	5,6	1,400	1,750
10	7,0	1,540	1,925	10	7,0	1,750	2,188
12	8,4	1,848	2,310	12	8,4	2,100	2,625
15	10,5	2,310	2,888	15	10,5	2,625	3,281
18	12,6	2,772	3,465	18	12,6	3,150	3,938
20	14,0	3,08	3,850	20	14,0	3,500	4,375
22	15,4	3,388	4,235	22	15,4	3,850	4,813
25	17,5	3,850	4,813	25	17,5	4,375	5,469

Design Strength p_w of Fillet Welds

Steel Grade	Electrode Classification		
	35	43	50
	N/mm^2	N/mm^2	N/mm^2
S275	220	220	220
S355	220	250	250
S460	220	250	280

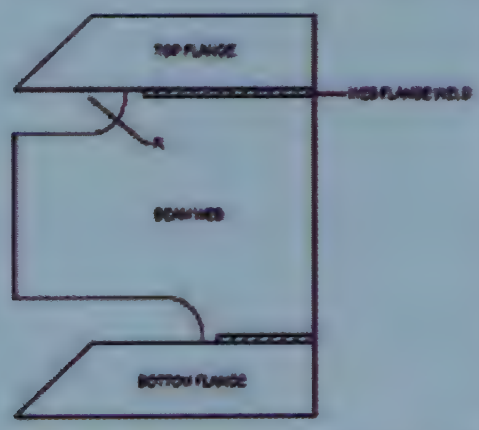
3.2.8 F-Weld Access

i) **Coping Hole:** The weld access hole or rat hole is a structural engineering technique in which a part of the web of an I-beam or T-beam is cut out at the end or ends of the beam.

The hole in the web allows a welder to weld the flange to another part of the structure with a continuous weld the full width on both top and bottom sides of the flange. Without the weld access hole, the middle of the flange would be blocked by the web and inaccessible for welding.

The hole also minimizes the induction of thermal stresses with a combination of partially releasing the welded section, avoiding welding the T section where the flange joins the web and improving cooling conditions.

The configuration adopted for web access holes affects how the beam joint will bend when under stress.



Side view of a beam with two weld access holes cut into the web. Also the flanges have been beveled in preparation for welding.

Fig. 23 Coping Hole

ii) **SMAW:** Where weld access is available at the member end, the clearance is independent of the flange width, and the angle between the electrode and the longitudinal weld axis is critical, where a 20° minimum electrode angle is recommended for straight electrodes.

If the proper clearance cannot be attained, the more expensive alternative can be used. Also, a single-sided fillet weld may be acceptable in some cases. Because these joints result in a crevice, corrosion can cause problems for structures exposed to weather unless measures are taken to prevent moisture accumulation within the joint.

An additional parameter that may affect weld clearance requirements is the "banking" of weld metal to counteract the effect of gravity for welds made in the horizontal position. To obtain equal-leg fillet welds, the welder typically rotates the electrode toward the horizontal surface, so the arc is directed more toward the vertical surface.

Based on this, a vertical obstruction may be more critical than a horizontal obstruction in the welding of joints in the horizontal position.

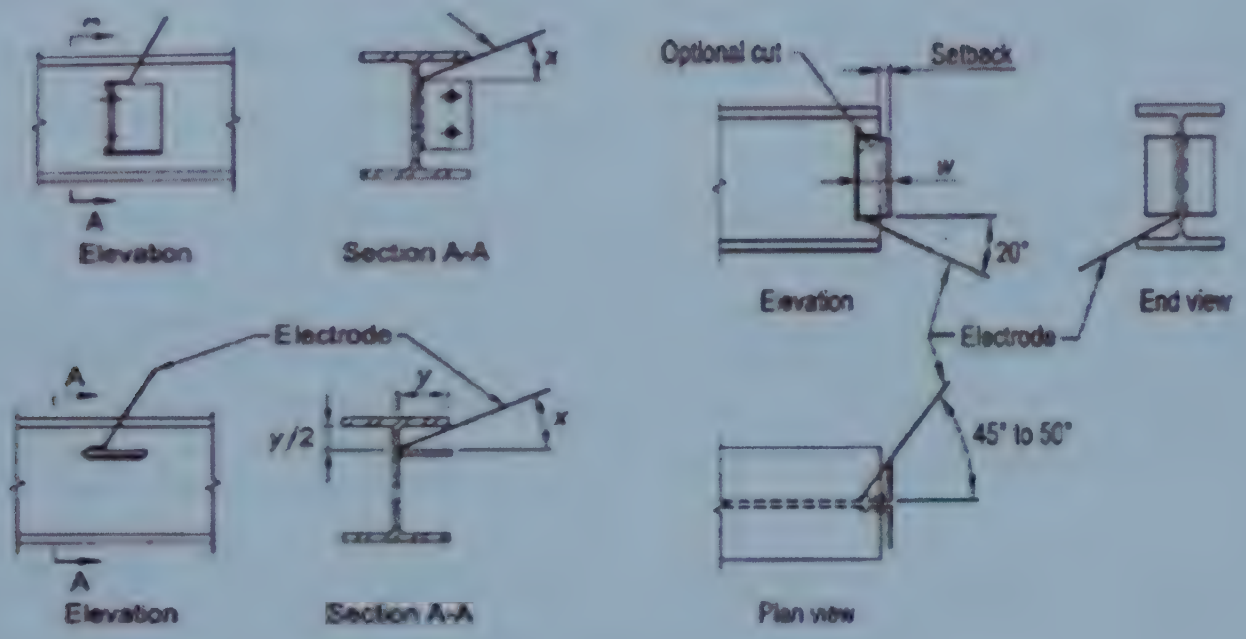


Fig. 24 Details of SMAW

iii) **GMAW/FCAW:** A primer flux cored arc welding (FCAW) is the most common process for welding steel structures. Most shops now use gas-shielded flux cored arc welding (FCAW-G) or gas metal arc welding (GMAW) in production, and self-shielded flux-cored arc welding (FCAW-S) is the most- used process for field welding. The geometry of a FCAW or GMAW welding gun is much different from that of a SMAW "stick" electrode. Additionally, electrode manipulation techniques may be different between the processes.

Recommended clearances for FCAW and GMAW welding, based on the research, are shown in **Fig. 25**

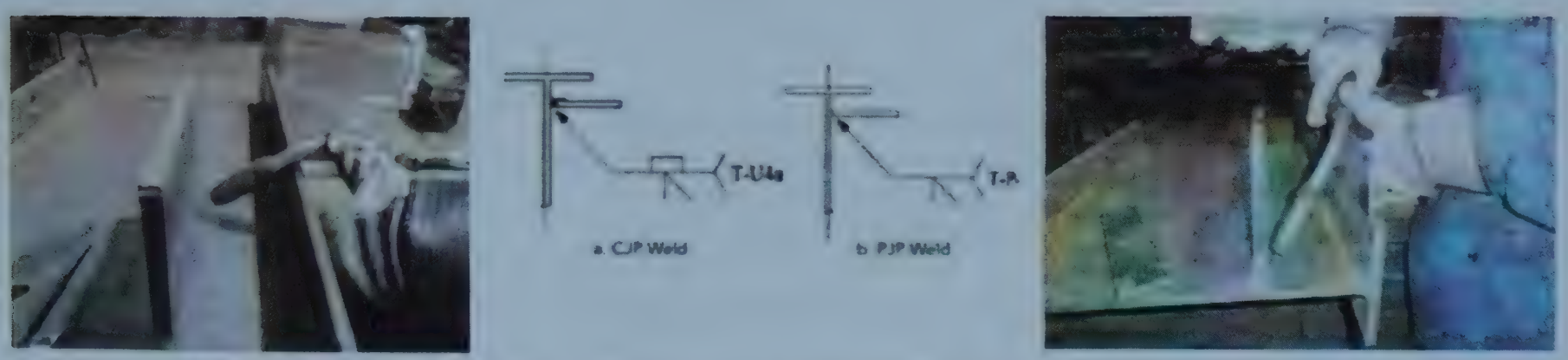


Fig. 25 Detail of GMAW/ FCAW

Two joint geometries are considered (as illustrated in **Fig. 26**)

1) Case 1 – where the welded element is parallel to the obstruction and

2) Case 2 – where the welded element is perpendicular to the obstruction.

Case 1: When welding near wide obstructions, (large b-dimension), the welder's hand and the welding gun must fit into the opening while allowing enough room for proper electrode manipulation. For this geometry, the required clearance, c_{min}, is the minimum of $b/2$ and 100 mm. This should be considered as the absolute minimum clearance. Since with the SMAW process, a 30° electrode angle is recommended for optimum production efficiency, it results in the $0.6b$ clearance requirement.

Case 2: This configuration does not require the entire welding gun to fit within the clearance dimension, allowing less restrictive clearances. Due to the high heat input with the FCAW and GMAW processes and the increased access between the gas nozzle and the weld surface. c_{min} is dependent on the plate thickness, t. If these requirements cannot be met, the welded edge can be bevelled.

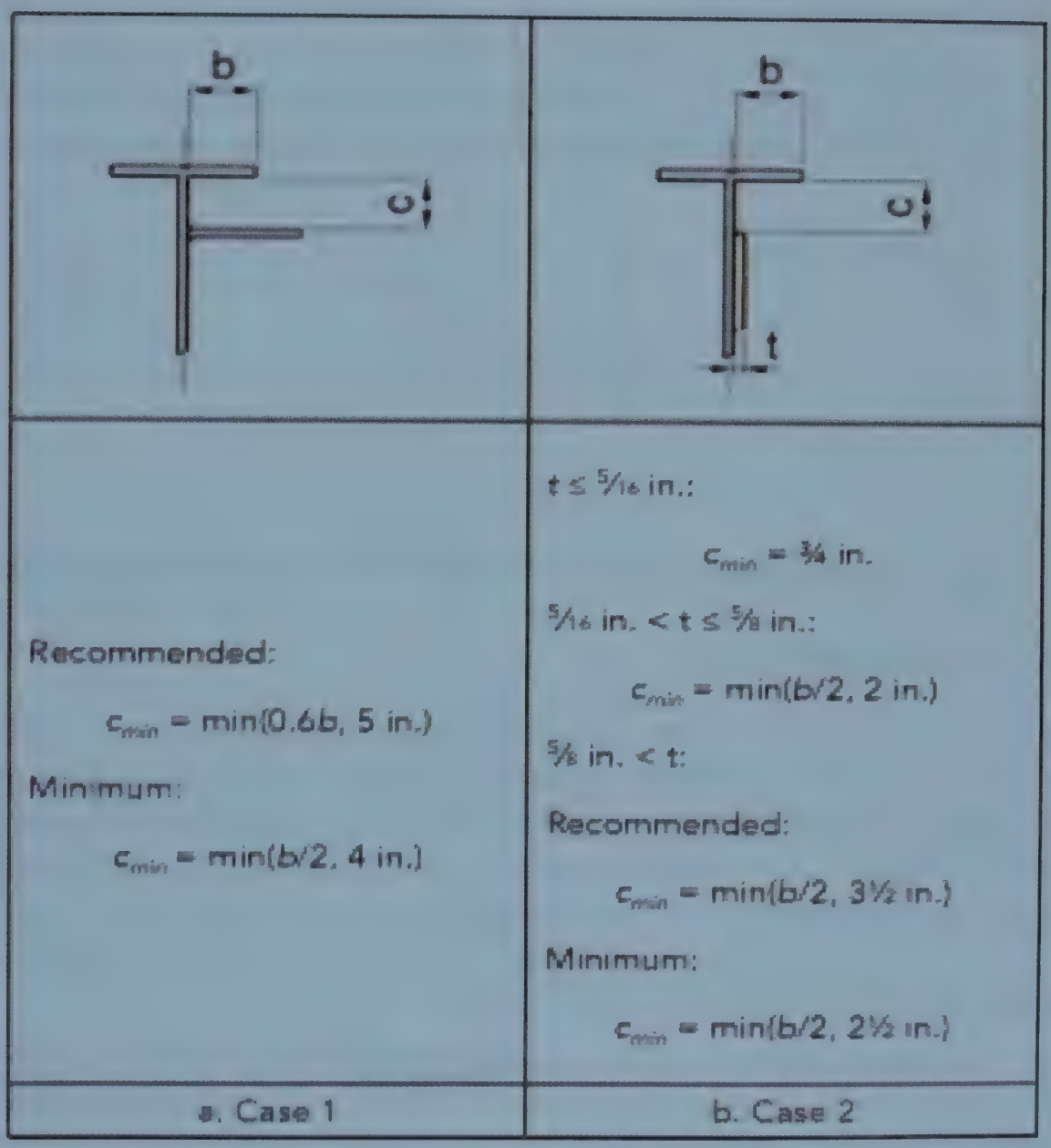


Fig. 26 Detail of Joint Geometries

The preparation must provide adequate access so the arc can be directed against the base metal. **Fig. 27a** shows a tee joint with a square groove preparation that is not prequalified because the arc cannot be directed against the base metal. A similar detail is shown for a corner joint in **Fig. 27b**. For relatively thin materials, the corner joint is prequalified because arc access is not obstructed as it is for the tee joint. A prequalified tee joint with a single-bevel weld preparation is shown in **Fig. 27c**, where the arc can be easily directed against the bevelled surface.

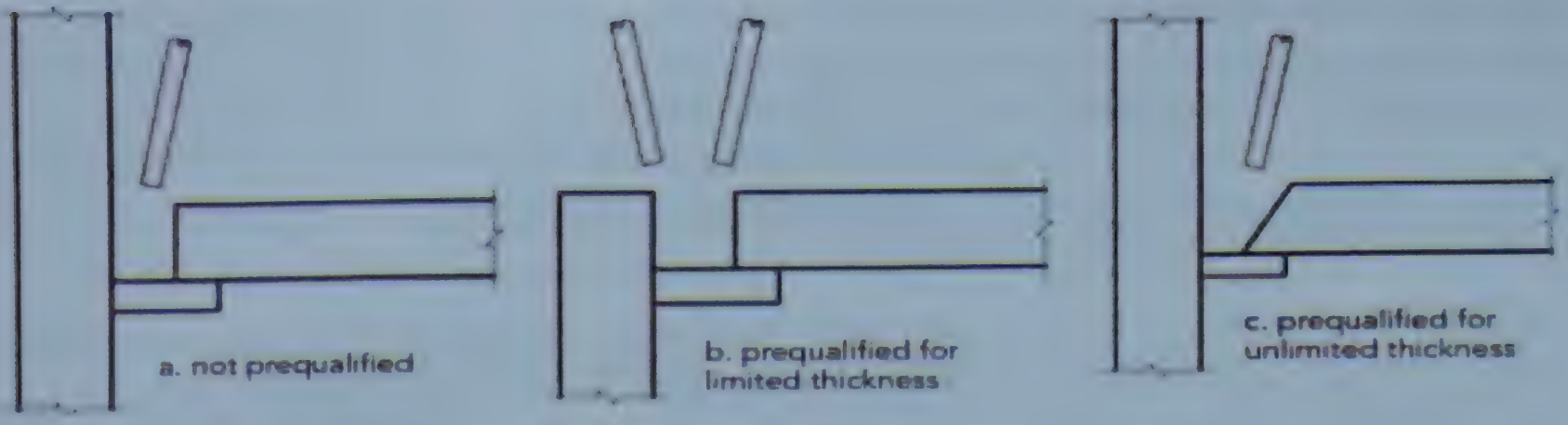


Fig. 27 Preparation of Joints

A potential access problem for groove-welded joints is shown in **Fig. 28**, where a plate is welded to the web of an I-shape member. In many cases, the arc cannot be properly directed against the base metal due to an obstruction caused by the flange. Also, there is not a direct line of sight for the welder or the inspector to view the bevelled weld preparation surface.

Fig. 28 Potential Access Problem for Groove Welded Joints

3.2.9 Welding Procedure Specification

A Welding Procedure Specification (WPS) is a document that serves as a guide for the effective creation of a weld that meets all applicable code requirements and production standards. WPS is component specific. The procedure takes into account accessibility of the weld bead, the type of welding i.e., SMAW/ GTAW/ FCAW/ SAW, welding parameters such as voltage, current, type of flux/ gas, trolley speed, wire feed, stick out etc., sequence of welding.

3.2.10 Precaution against Weld Defects / Distortions

1) **Weld Crack:** The most serious type of welding defect is a weld crack and it's not accepted almost by all standards in the industry. It can appear on the surface, in the weld metal or the area affected by the intense heat.

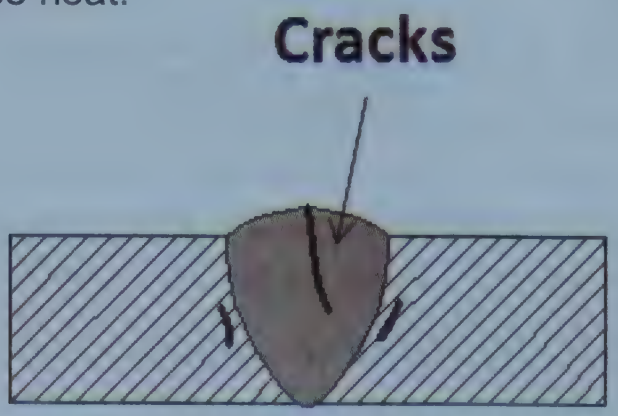


Fig. 29 Weld Crack

There are different types of cracks, depending on the temperature at which they occur:

i) **Hot Cracks:** These can occur during the welding process or during the crystallization process of the weld joint. The temperature at this point can rise over 1000°C.

ii) **Cold Cracks:** These cracks appear after the weld has been completed and the temperature of the metal has gone down. They can form hours or even days after welding. It mostly happens when welding steel. The cause of this defect is usually deformities in the structure of steel.

iii) **Crater Cracks:** These occur at the end of the welding process before the operator finishes a pass on the weld joint. They usually form near the end of the weld. When the weld pool cools and solidifies, it needs to have enough volume to overcome shrinkage of the weld metal. Otherwise, it will form a crater crack.

Causes

i) Use of hydrogen when welding ferrous metals.
ii) Residual stress caused by the solidification shrinkage.
iii) Base metal contamination.
iv) High welding speed but low current.
v) No preheat before starting welding.
vi) Poor joint design.
vii) A high content of sulphur and carbon in the metal.

Remedies

i) Preheating of the metal, as required.
ii) Provision for proper cooling of the weld area.
iii) Use of proper joint design.
iv) Removal of impurities.
v) Use of appropriate metal.
vi) Ensuring to weld a sufficient sectional area.
vii) Use of proper welding speed and amperage current.
viii) To prevent crater cracks, it is to be ensured that the crater is properly filled.

2) **Porosity:** Porosity occurs as a result of weld metal contamination. The trapped gases create a bubble-filled weld that becomes weak and can with time collapse.

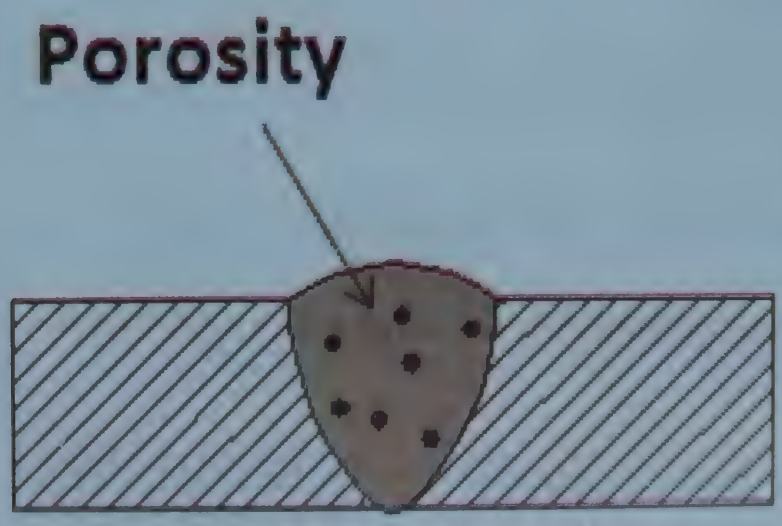


Fig. 30 Porosity in Weld

Causes

i) Inadequate electrode deoxidant.
ii) Using a longer arc.
iii) The presence of moisture.
iv) Improper gas shield.
v) Incorrect surface treatment.
vi) Use of too high gas flow.
vii) Contaminated surface.
viii) Presence of rust, paint, grease, or oil.

Remedies

i) Cleaning the materials before commencement of welding.
ii) Use of dry electrodes and materials.
iii) Use of correct arc distance.
iv) Checking the gas flow meter to make sure that it is optimized as required with proper with pressure and flow settings.
v) Reducing arc travel speed, which will allow the gases to escape.
vi) Use of right electrodes.
vii) Use of a proper weld technique.

3) **Undercut:** This welding imperfection is the groove formation at the weld toe, reducing the cross- sectional thickness of the base metal. The result is the weakened weld and work piece.

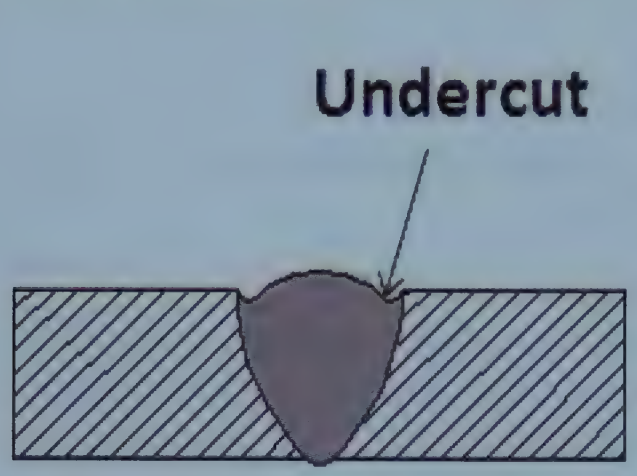


Fig. 31 Undercut in Weld

Causes

i) Too high weld current.
ii) Too fast weld speed.
iii) The use of an incorrect angle, which will direct more heat to free edges.
iv) The electrode is too large.
v) Incorrect usage of gas shielding.
vi) Incorrect filler metal.
vii) Poor weld technique.

Remedies

i) Use of proper electrode angle.
ii) Reducing the arc length.
iii) Reducing the electrode's travel speed, but it also should not be too slow.

iv) Choice of shielding gas with the correct composition for the material type you'll be welding:
v) Use of proper electrode angle, with more heat directed towards thicker components.
vi) Use of proper current, reducing it when approaching thinner areas and free edges.
vii) Choice of a correct welding technique that doesn't involve excessive weaving.
viii) Use of multi pass technique.

4) **Incomplete Fusion:** This type of welding defect occurs when there's a lack of proper fusion between the base metal and the weld metal. It can also appear between adjoining weld beads. This creates a gap in the joint that is not filled with molten metal.

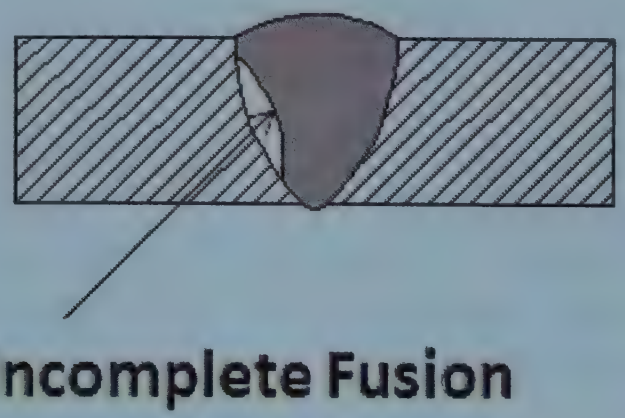


Fig. 32 Incomplete Fusion in Weld

Causes

i) Low heat input.
ii) Surface contamination.
iii) Electrode angle is incorrect.
iv) The electrode diameter is incorrect for the material thickness you're welding.
v) Travel speed is too fast.
vi) The weld pool is too large, and it runs ahead of the arc.

Remedies

i) Use of a sufficiently high welding current with the appropriate arc voltage.
ii) Before commencement of welding, cleaning the metal.
iii) Avoiding molten pool from flooding the arc.
iv) Use of correct electrode diameter and angle.
v) Reducing deposition rate.

5) **Incomplete Penetration:** Incomplete penetration occurs when the groove of the metal is not filled completely, meaning the weld metal doesn't fully extend through the joint thickness.

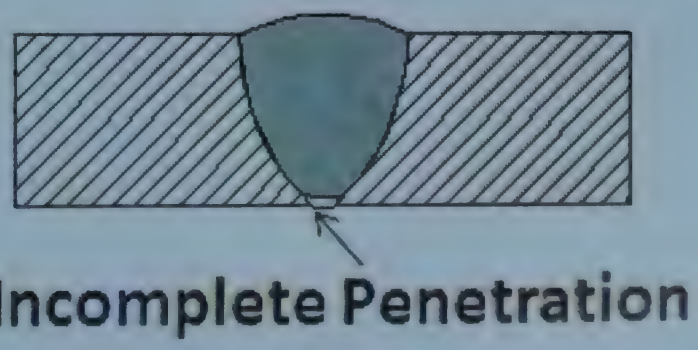


Fig. 33 Incomplete Penetration of Weld

Causes

i) There was too much space between the metal you're welding together.
ii) Moving the bead too quickly, which does not allow enough metal to be deposited in the joint.
iii) Using a too low amperage setting, which results in the current not being strong enough to properly melt the metal.
iv) Large electrode diameter.
v) Misalignment.
vi) Improper joint.

Remedies

i) Use of proper joint geometry.
ii) Use of a properly sized electrode.
iii) Reducing arc travel speed.
iv) Choice of proper welding current.
v) Checking for proper alignment.

6) **Slag Inclusion:** Slag inclusion is one of the welding defects that are usually easily visible in the weld. Slag is a vitreous material that occurs as a by-product of stick welding, flux-cored arc welding and submerged arc welding. It can occur when the flux, which is the solid shielding material used when welding, melts in the weld or on the surface of the weld zone.

Fig. 34 Slag Inclusion in Weld

Causes

i) Improper cleaning.
ii) The weld speed is too fast.
iii) Not cleaning the weld pass before starting a new one.
iv) Incorrect welding angle.
v) The weld pool cools down too fast.
vi) Welding current is too low.

Remedies

i) Increase current density.
ii) Reduce rapid cooling.
iii) Adjust the electrode angle.
iv) Remove any slag from the previous bead.
v) Adjust the welding speed.

7) **Spatter:** Spatter occurs when small particles from the weld attach themselves to the surrounding surface. It is an especially common occurrence in gas metal arc welding. It cannot be completely eliminated, however, there are a few ways it can be kept to a minimum.

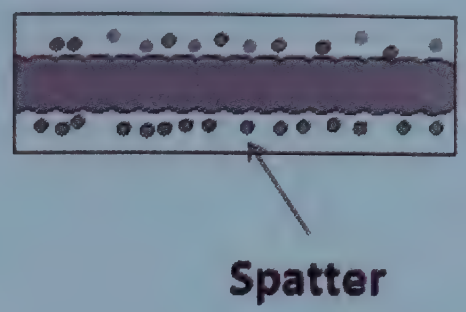


Fig. 35 Spatter in Weld

Causes

i) The running amperage is too high.
ii) Voltage setting is too low.
iii) The work angle of the electrode is too steep.
iv) The surface is contaminated.
v) The arc is too long.
vi) Incorrect polarity.
vii) Erratic wire feeding.

Remedies

i) Cleaning surfaces prior to welding.
ii) Reducing the arc length.
iii) Adjusting the weld current.
iv) Increasing the electrode angle.
v) Use of proper polarity.
vi) Making sure there is no feeding issues.

8) **Distortion:** Once a molten weld bead has been deposited and begins to cool, it will solidify and attempt to contract, both along and across its axis. This will induce residual tensile stresses.

Types of Distortion

i) Longitudinal shrinkage of a built-up welded member can cause slender webs and outstands to buckle and
ii) If the weld or welds are eccentric to the centroid of the member cross-section, it will cause overall bowing of the member.
iii) Transverse shrinkage is likely to cause both angular and out-of-plane distortions.

Causes

i) Restraint imposed by the surrounding structure.
ii) Lack of complete rigidity in the surrounding structure.

Remedies

i) Reverse bending of flanges during assembly.
ii) Strengthening of flanges by clamping.
iii) Application of heat at the joint of flange and web.
iv) Hydraulic lifting of the edges of flange after welding.

v) Hydraulic jacks and fixtures for mechanical straightening.
vi) Application of heat and mechanical straightening.

3.2.11 Precaution for On-Site Welding

Usually, the members were assembled through field joints by riveting or bolting. As the time progressed the field joints were being done either by bolting or by welding. The science of welding and welding consumables have improved over time and have proven their performance in field joints too.

The welding at site needs following precautions:

i) The field joint preparation prior to welding.
ii) Qualification of welders.
iii) Use of proper welding power source.
iv) Availability of stable power supply at site.
v) Precautions to protect weld joint, immediately after welding, from rainwater.
vi) Precautions to protect against high wind flow during welding.
vii) Using right kind of consumables.

If welding is done with adequate precaution and with right selection of consumables, the field joints will perform satisfactorily.

3.2.12 Field Joint Preparation

Field joint area should be cleaned of all rusts and paints. This can be done by sand blasting, if permissible and possible. Normally, field joints are fillet joints. Prior to assembly of the cover/splice plate a coat of paint is applied below the joint, it should be ensured that the portion of the joint that is likely to be heat affected, is cleaned from the freshly applied paint.

All protective paints, applied after fabrication has to be removed from the welding joint area. The other methods of joint cleaning are by mechanical wire brush and manual wire brush. If possible, the area can be chemically cleaned by any cleaning reagents such as dilute nitric acid.

Welding should commence immediately after cleaning so that the rusting does not set in.

3.2.13 Qualification of Welders

Using qualified welders to do the field joints is vital to proper welding. The welder should be qualified as per relevant IS code. The field joints are normally fillet joints but if butt weld (U/V) joints has been specified by the designer, the welders have to be qualified for that too.

The welder needs to be qualified for all three positions - 1F, 2F and 3F.

If butt joint is adopted, then the welder needs to be qualified for 1G, 2G and 3G positions.

3.2.14 Welding Power Source

All welding power source should be 3 phase DC power source with proper amperage. The requirement of amperage should be ascertained from the specification of welding rod for MMAW process, flux cored welding wire for GMAW / FCAW process. Usually, the current depends on the diameter of the welding consumable (welding rod/ welding wire). Right size of welding cable and earth cable should be used. Using lesser diameter welding cables will result in cable heating and

discomfort to the welder. Using lesser diameter earth cable can result in inadequate earth for the job and cause complications.

The welding power source should be as near to the weld joint as possible. Providing adequate length of power cable to take the welding power source as near to the joint as possible, is recommended. Use of long welding cable for covering the distance from the power source to weld point should be avoided.

Supply of stable power is vital to good welding. Welding needs power supply with a stable voltage. High voltage fluctuations may result in improper arcing of the rod and result in incomplete weld penetration and lack of fusion. Low voltage may damage the power source.

3.2.15 Protection against Rain

Welding is a fusion process. The metal to be joined and the welding rod melts at a temp near to melting point of steel of 1600°C.The weld metal pool undergoes rapid cooling after welding completion. If the welding joint comes in contact with water from rainwater before complete cooling of the welds, the weld joint may result in cracking. The repair procedure will involve complete removal of the weld by grinding and rewelding involving a lot of cost, time, and poor joint.

Adequate precaution should be taken by covering weld area by temporary overhead cover arrangement so that the welder has access to do the welding and adequate cover for rain is also ensured. The protection will result in welding during night and give protection against night dew.

3.2.16 Precaution against Wind

Adequate protection against blowing wind-flow is essential for good welding. The wind is normally encountered at higher velocity when the welding is carried out at an elevated place after assembly of the girder above the ground over staging.

The effect of wind is not pronounced in MMAW process as the flux, on top of the wire, guides the arc and it melts during the welding process and covers the weld metal pool which results in giving protection to the weld bead against corrosion.

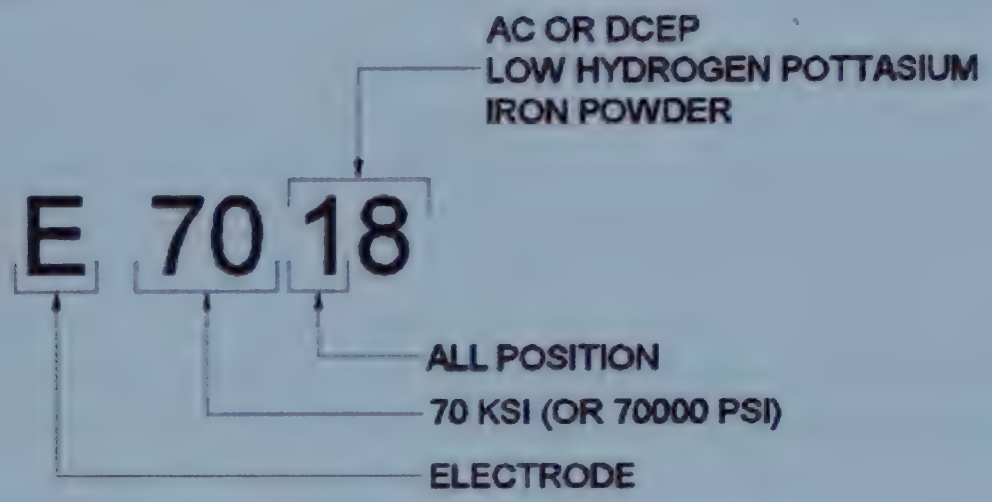
The effect of wind is pronounced in case of GMAW / FCAW process. The shielding gas CO_2 / Argon Carbon dioxide mixture, as per the WPS, can fall short at the weld joint area by the blowing wind. This results in porosity and extremely poor quality of weld. Therefore, the welding area should have sufficient side cover shield so that the welding area is adequately protected against blowing wind.

3.2.17 Selection of Welding Consumable

Properties	E7018 (Welding Consumable)	E4102 (Raw Steel)
Chemical		
C	0.1	0.2
Mn	1.65	1.6
Si	0.46	0.45
P	0.03	0.045
S	0.013	0.045
Cu	0.10	
Physical		
Yield	487 MPa	390 MPa
Ultimate	613 MPa	540 MPa
Elongation	28.3mm	20mm
Gauge Length	4d	5.65d
CVN	28 Joules	25 Joules

3.2.18 Examples of Most Used Welding Rods

i) **E6010** = High Cellulose Sodium flux, DCEP (direct current electropositive) (the electrode is connected to positive terminal) for flat, vertical, overhead & horizontal position.
ii) **E6011** = High Cellulose Potassium flux, AC or DCEP for flat, vertical, overhead & horizontal position.
iii) **E6012** = High Titania Sodium flux, AC or DCEN (direct current electronegative) (the electrode is connected to negative terminal) for flat, vertical, overhead & horizontal position.
iv) **E6013** = High Titania Sodium flux, AC or DCEP for flat, vertical, overhead & horizontal position.
v) **E7018** = Low Hydrogen Potassium Iron powder Flux AC or DCEP for flat, vertical, overhead & horizontal position.



E7018 = Low Hydrogen Potassium Iron powder Flux AC or DCEP for flat , Vertical, Overhead & Horizontal position.

Fig. 36 Solid Welding Rod Nomenclature

E70C-6

This is a general-purpose Flux Cored Wire.
The number "**7**" in the classification refers to tensile strength in Psi , multiplied by 10,000 (instead of 1,000 in the case of a non-cored MIG wire).
"**0**" means the wire is only good for Horizontal or Flat Welding.
"**C**" – Denotes cored wire
"**6**" – This denotes the deoxidizing agent and cleansing agent on the electrode. This is a copper coating on the electrode and there are different types too.

E71T-8

The number " 7 " in the classification refers to tensile strength in Psi , multiplied by 10,000 (instead of 1,000 in the case of a non-cored MIG wire).
T for tubular.
The digit 1 indicates the wire can be used in all positions.
The "8" signifies Low Hydrogen which calls to mind the common stick electrode E7018.

Fig. 37 Flux Cored Wire Nomenclature

ER 70 S-6

The letters and numbers represent the following:

ER - An Electrode or filler rod that is used in a MIG wire feed

70 - A minimum of 70,000 pounds of tensile strength per square inch of weld

S - Solid Wire

6 - The amount of deoxidizing agent and cleansing agent on the electrode. This is a copper coating on the electrode .

There are other types of Coating too.

Welding Carbon Steels and Mild Steels

AWS Electrode Nomenclature : ER70s-6/E70C-6/E71T-8

Shielding Gas : C25 (25%Carbon Dioxide & 75% Argon) or 100 % CO_2

Fig. 38 Solid Mig Wire Nomenclature

***Notes**: Much of the Nomenclature used above and referred to in the field, are AWS nomenclature. For finding the equivalent nomenclature in BIS, IS: 814 may be referred to.*

4 CONNECTION DETAILING

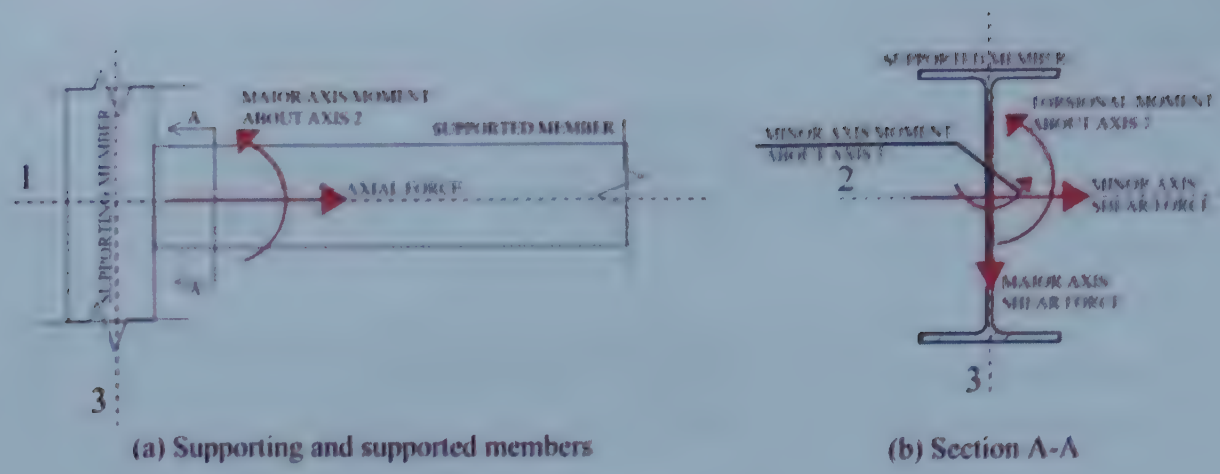
In this chapter, structural steel detailing is described with respect to the forces transferred at connection locations. Members which are joined at connections are classified as supporting and supported members, as defined in **Fig. 39a**. It is to be noted that in splice connections, supporting and supported members are interchangeable.

Connection force at the ends of members can be obtained from an analysis on the entire structure (refer **Section 5** for example steel bridge structural systems). Care is to be taken to ensure that the structural analysis model is consistent with the as-detailed and as-built conditions (or temporary / erection condition where applicable).

For example, if the structural analysis model includes a fixed moment-resisting connection at any location, the connection should be detailed in such a way that the corresponding fixity is maintained at service / ultimate loads as applicable (refer 4.1.3 for moment connections). Conversely, if a connection is detailed as a “simple connection” or “shear-only” connection, it should be capable of developing moment releases by local plasticity at bolt holes in simple shear connections, or by using short-slotted holes at the shear- force transfer joints (refer 4.1.1 for shear connections).

Each structural joint may be required to transfer one or more out of three components of force plus three components of moment, as shown in **Fig. 39**. Shear forces act either along the major axis (axis 3 in **Fig. 39b**) or along the minor axis (axis 2 in **Fig. 39b**) of the supported member. Axial forces develop along the axis of the member (axis 1 in **Fig. 39a**). Major axis moments act about the major axis of the supported member (refer **Fig. 39a**) and minor axis moments act about the minor axis of the supported member (axis 3 in **Fig. 39b**). Torsional moments act about the axis of the supported member (refer **Fig. 39b**).

Examples of members transferring each type of loads at its ends (shear forces, axial forces and bending moments) are noted at the beginning of each section. The transferred forces are referred in the local coordinate system of the supported member, as defined in **Fig. 39**.



(a) Supporting and supported members (b) Section A-A

Fig. 39 Coordinate System and Force Component Definitions

In any connection detail, the supporting member is to be designed for the forces arising from the supported connection (in addition to any other forces). Load effects on supporting and supported members are equal and opposite in splice connections.

It should be noted that the guidelines provided in the example load paths described in this chapter are typically conservative. Assumed load paths that are different from the ones indicated herein are acceptable, provided they are justified by acceptable engineering rationalization.

4.1 Bolted Connections – General Detailing

General bolt details, bolt action, sizes, edge distances and bolt holes are discussed in **Section 3**. This section describes load paths for various bolted connection details.

4.1.1 Detailing for Shear Force

1) Detailing for Major Axis Shear Force

Some examples of major axis shear force transfer are at bearing connections at the interface of a pier, connection between bottom horizontal and verticals in a bowstring girder, or any splice connection at non-zero shear force locations. Major axis shear force transfer can occur in the gravity direction (predominant) or in the uplift direction (opposite to gravity).

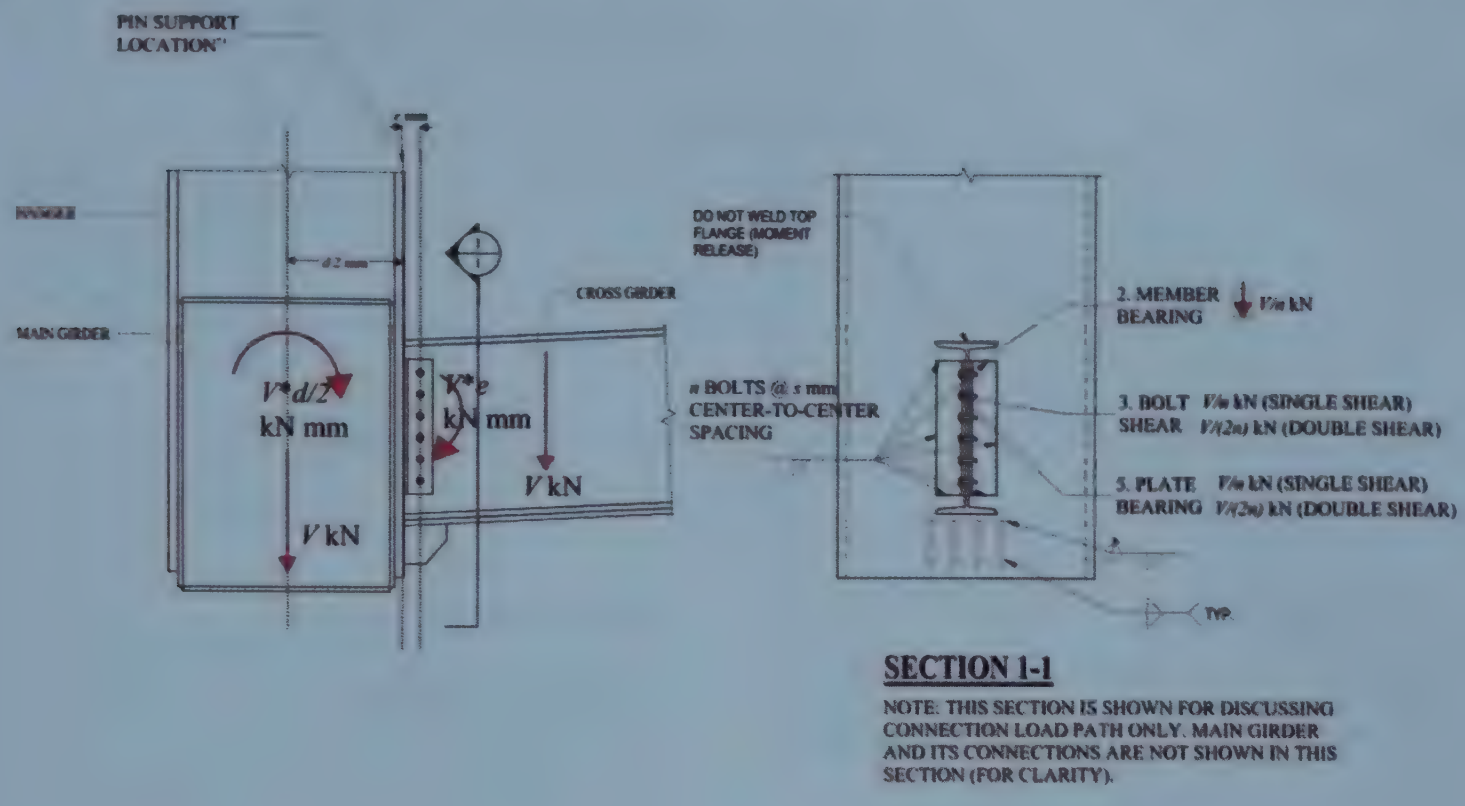
A representative load path for major axis shear force transfer is shown in **Fig. 40**. Various ultimate limit states to be checked for this connection detail are listed. Against each limit state, references to the governing codal provisions are included, which is mostly self-explanatory.

Limit state 4 in **Fig. 40**, refers to eccentrically loaded bolt group capacity. The bolt group is subjected to a combination of shear and moment due to eccentricity from the support location as shown. The resultant shear due to the combined loads may be calculated using conservative elastic vector analysis, or by a rationalized plastic method based on the instantaneous center of rotation.

In the example shown in **Fig. 40**, the shear force in the supported member creates an axial force demand on the hanger.

It should be ensured that the as-detailed condition for any connection is analysed during modelling and connection design, including the effect of partial fixity. For example, in a simple shear connection as shown in **Fig. 40**, partial fixity created due to multiple bolt rows may result in additional stresses. It should be ensured that this partial fixity does not lead to cracking, either by

ductile release of the fixity via local plasticity, or by maintaining the stresses within allowable limits.



SECTION 1-1

NOTE: THIS SECTION IS SHOWN FOR DISCUSSING CONNECTION LOAD PATH ONLY. MAIN GIRDER AND ITS CONNECTIONS ARE NOT SHOWN IN THIS SECTION (FOR CLARITY).

1. SHEAR FORCE AND BENDING MOMENT CAPACITY AT NET SECTION
(Refer IRC:24-2010 Section 509)

2. MEMBER BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Section 512.5)

3. BOLT SHEAR CAPACITY (SINGLE OR DOUBLE)*2
(Refer IRC:24-2010 Section 512.5)

4. ECCENTRICALLY LOADED BOLT GROUP CAPACITY

5. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Section 512.5)

6 PLATE NET SECTION SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)

NOTES:
*1) REMAINING ELEMENTS IN-BETWEEN PIN SUPPORT LOCATION AND CENTERLINE OF SUPPORTING MEMBER (INCLUDING PLATE, BOLTS, WELDS AND SUPPORTING MEMBER) ARE TO BE DESIGNED FOR MOMENT DUE TO ECCENTRICITY OF SHEAR FORCE TRANSFER. NOTE THAT THIS MAY ACT AS A TORSIONAL OR MINOR AXIS MOMENT ON THE SUPPORTING MEMBER, DEPENDING UPON THE CONFIGURATION.

*2) FOR BOLTED CONNECTIONS WHERE SLIP IS NOT PERMITTED, BOLT SHEAR CAPACITY IS LIMITED BY FRICTION GRIP SLIP CAPACITY (REFER IRC:24-2010 Section 512.6)

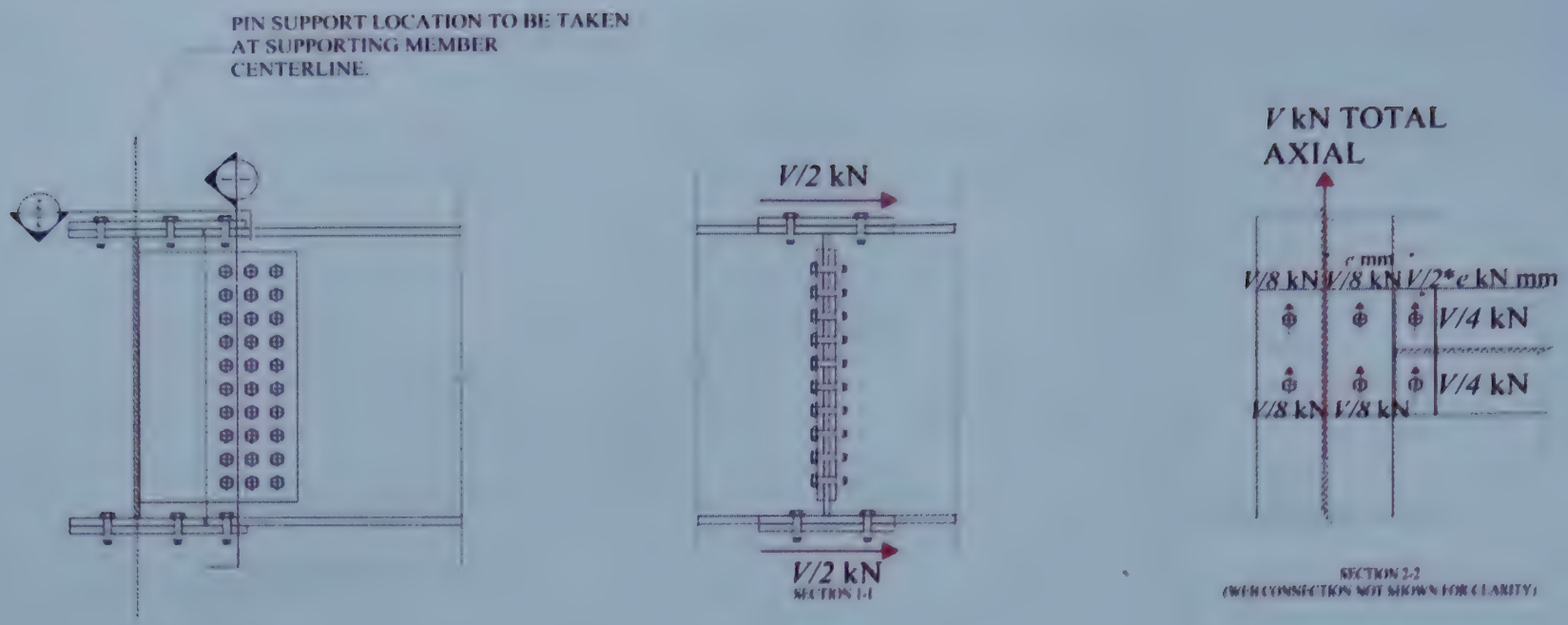
3) PLATE ELEMENTS OF CROSS SECTIONS IN THE LOAD PATH ARE TO BE CHECKED FOR LOCAL INSTABILITIES PER IS 800 2007 Section 3.7 AND OTHER RELEVANT CODAL PROVISIONS

Fig. 40 Load Path Detailing Considerations for Major Axis Shear Force Transfer (Bearing Type Bolts)

2) Detailing for Minor Axis Shear Force

Shear force transfer in the minor axis direction of the supported member (refer **Fig. 39**) may be required to resist forces perpendicular to member axis, such as in wind girts. Weak axis shear may also arise from supports to axial force carrying members. For example, in **Fig. 42**, weak axis shear force is applied on the supporting member, as a result of axial force in the supported member.

Minor axis shear may be transferred through beam flanges, as described in **Fig. 41**. It is not desirable to transfer minor axis shear through beam webs since it puts the bolts in tension and the shear plate in weak axis flexure.



AT SUPPORTED MEMBER

1. MINOR AXIS SHEAR FORCE CAPACITY FOR MEMBER
(Refer IRC:24-2010 Section 509 considering minor axis shear area)

2. MEMBER BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Section 512.5)

3. BOLT SINGLE SHEAR CAPACITY (IF SINGLE FLANGE PLATE IS USED)
(Refer IRC:24-2010 Section 512.5, see Note 2 on Bolted Shear Connection Detail)

4. ECCENTRICALLY LOADED BOLT GROUP CAPACITY

5. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Section 512.5)

6. PLATE NET SECTION SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)

AT SUPPORTING MEMBER

11. BOLT BEARING ON SUPPORTING MEMBER CAPACITY
(Refer IRC:24-2010 Section 512.5)

12. SUPPORTING MEMBER SECTION AXIAL CAPACITY
(Refer IRC:24-2010 Sections 506 and 507)

Fig. 41 Load Path Detailing Considerations for Minor Axis Shear Force Transfer (Bearing Type Bolts)

4.1.2 Detailing for Axial Force

Some examples of axial force members are hanger rods in bowstring girder bridges, diagonals in truss bridges and horizontal or vertical bracing members.

Axial force may be transferred in member webs, or in one or both of its flanges, depending upon relative proportions of forces and interaction capacities. Load path detailing for axial force transfer through member flanges is described in **Fig. 42**. If axial force is transferred through the web, consideration should be made for interaction limit states (refer 4.1.4). An example load path for axial force resisting connection to bracing members is shown in **Fig. 54**.

As per IS : 800-2007, axial force carrying tension members are to be checked for gross section yielding, net section rupture and block shear. Axial load carrying compression members are to be checked for buckling instabilities for each element, as discussed in 4.6 on Gusset Plates.

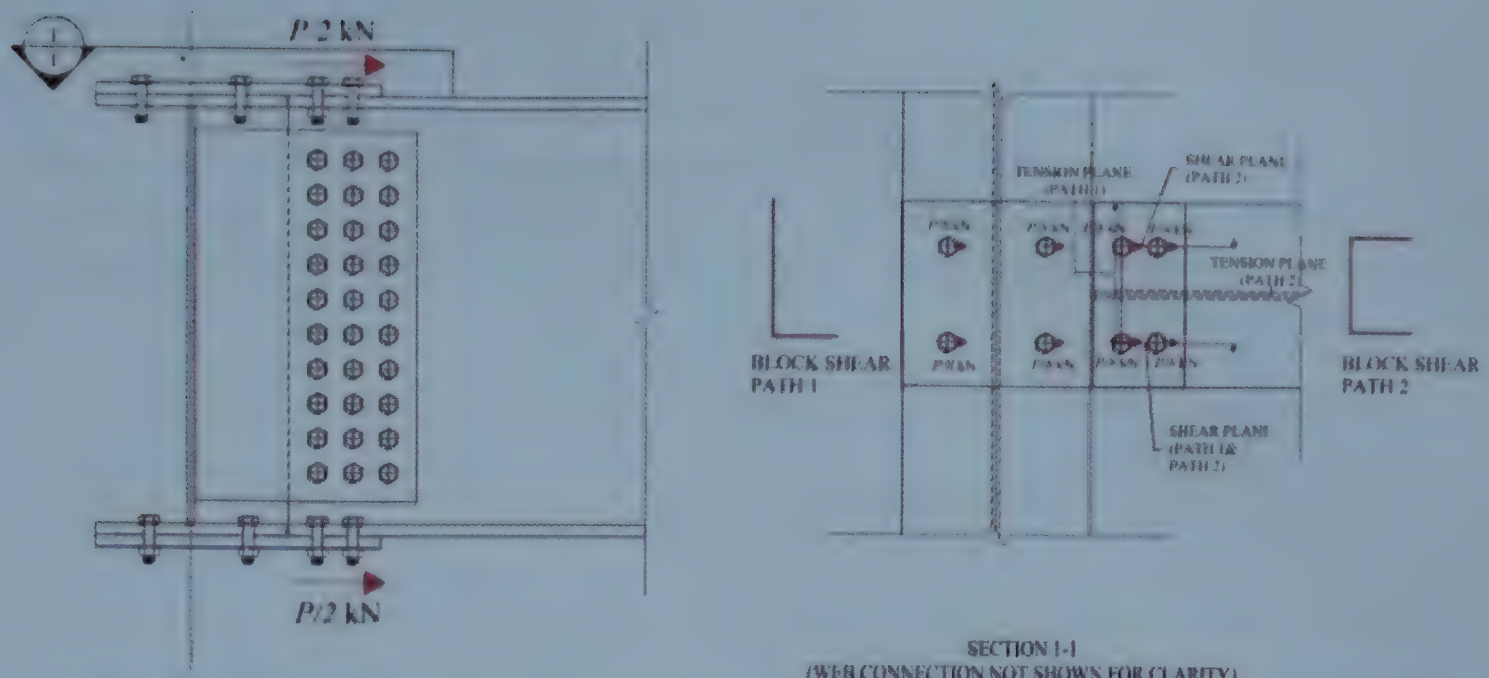

P/2 kN
P/2 kN
TENSION PLANE (PATH 1)
SHEAR PLANE (PATH 2)
TENSION PLANE (PATH 2)
BLOCK SHEAR PATH 1
BLOCK SHEAR PATH 2
SHEAR PLANE (PATH 1& PATH 2)
SECTION 1-1
(WEB CONNECTION NOT SHOWN FOR CLARITY)
AT SUPPORTED MEMBER
1. AXIAL FORCE CAPACITY OF MEMBER
(Refer IRC:24-2010 Sections 506 and 507)
2. MEMBER BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Section 512.5)
3. BOLT SINGLE SHEAR CAPACITY (IF SINGLE FLANGE PLATE IS USED)
(Refer IRC:24-2010 Section 512.5, see Note 2 on Bolted Shear Connection Detail)
4. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Section 512.5)
6. PLATE TENSILE CAPACITY (Refer IRC:24-2010 Section 506)
AT SUPPORTING MEMBER
7. BOLT BEARING ON SUPPORTING MEMBER CAPACITY
(Refer IRC:24-2010 Section 512.5)
8. SUPPORTING MEMBER WEAK AXIS SHEAR CAPACITY
(Refer IRC:24-2010 Section 509 considering minor axis shear area)

Fig. 42 Load Path Detailing Considerations for Axial Force Transfer (Bearing Type Bolts)

In **Fig. 42**, the top and bottom flange plates, being tension members, need to be checked for block shear. Block shear is a combined shear + tension failure mode. All possible paths that can undergo block shear are to be checked during design. In **Fig. 42**, two example paths that can possibly undergo block shear are shown. Path 1 has a single tension plane and a single shear plane, whereas path 2 has two shear planes and a single tension plane. Block shear planes are further shown in the context of gusset plate detailing in 4.6.

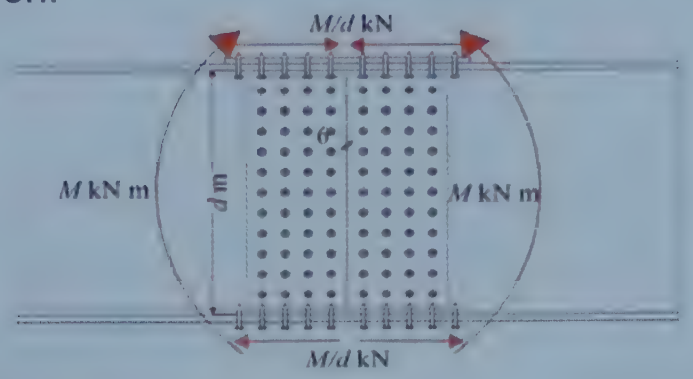
4.1.3 Detailing for Moment

1) Detailing for Major Axis Moment

Major axis moments arise in any rigid or semi-rigid connection within the structural system. Rigid connections do not permit rotational deformation under applied moment, and the angle between connected members remain the same before and after moment is applied. Semi-rigid connections permit finite rotation between connected members under applied moment according to the moment- rotation curve for the connection. Examples of moment connections include splice connections and at fixed ends of cantilever beams. It is important to ensure that the connection detail matches the assumptions made in the structural model. Any fixed connections in the structural model must be able to develop the design moment from the structural model. Conversely, any connections modelled as a moment release, should not develop unintentional moment fixity due to detailing.

An example load path for major axis moment transfer is shown in **Fig. 43**. In this example, bolts are assumed to be of bearing type. In practice, friction grip bolts are more commonly used in

moment connections, to prevent any slippage between plates. If friction grip bolts are used, shear forces between beam flange and flange plate are transferred through friction between plates. For such design, friction grip capacities (which are usually lower than bearing capacities) are to be used. In no case shall friction grip capacity and bearing capacity be summed together to resist the same load. As per Clause 512.6.3 of IRC:24, "When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection".



AT SUPPORTED ./ SUPPORTING MEMBER

1. MAJOR AXIS MOMENT CAPACITY FOR MEMBER
(Refer IRC:24-2010 Section 509)

2. BEAM FLANGE NET SECTION TENSION AND COMPRESSION CAPACITY
(Refer IRC:24-2010 Sections 506 and 507)

3. MEMBER BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Section 512.5)

4. BOLT DOUBLE SHEAR CAPACITY
(Refer IRC:24-2010 Section 512.5, see Note 2 on Bolted Shear Connection Detail)

5. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Section 512.5)

6. PLATE NET SECTION TENSION AND COMPRESSION CAPACITY
(Refer IRC:24-2010 Sections 506 and 507)

7. LOCAL STABILITY CHECKS ON SUPPORTED MEMBER AND PLATE
(Refer IS 800 2007 Section 3.7)

Fig. 43 Load Path Detailing Considerations for Major Axis Moment Transfer in Bolted Connections

2) Detailing for Torsion

Torsional moment may arise due to unbalanced moments on either side of a girder, such as due to a cantilever span on one side. Torsional moment may be transferred by resolving the moment as a force couple in two flanges, with each flange force acting in the minor axis direction, as described in **Fig. 44**. The supporting member to which the torsional force is transferred must possess adequate major axis moment capacity. The concentrated moment acting upon the supporting member is resolved as a small additional major axis shear reaction at the ends of the supporting member.

4.1.4 Detailing for Combined Effect

Combined effects may refer to any combination of three force components and three moment components described in **Fig. 39**. Combination of major axis moment plus major axis shear may be designed independently by transferring the major axis shear in the shear plate, and the moment resolved as an axial force couple in the flange plates, as described in **Fig. 45**.

Alternatively, part of the moment can be transferred through the beam web and shear plate (T beam action). In this case, part of the depth of the web plate is used to resist shear, and the remaining (at top and bottom) is considered for resistance to flexure. In case such assumptions are made, they

should be consistent with relevant design provisions, for example, low and high shear provisions in IS : 800-2007 **Section 8**. Stress interaction limit states must be properly accounted for wherever necessary.

AT SUPPORTED MEMBER

1. TORSION CAPACITY FOR MEMBER

2. MEMBER BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Sections 512.5)

3. BOLT SINGLE SHEAR CAPACITY (IF SINGLE FLANGE PLATE IS USED)
(Refer IRC:24-2010 Sections 512.5, see Note 2 on Bolted Shear Connection Detail)

4. ECCENTRICALLY LOADED BOLT GROUP CAPACITY

5. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Sections 512.5)

6. PLATE NET SECTION SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY (Refer IRC:24-2010 Sections 510)

AT SUPPORTING MEMBER

11. BOLT BEARING ON SUPPORTING MEMBER CAPACITY
(Refer IRC:24-2010 Sections 512.5)

12. SUPPORTING MEMBER MAJOR AXIS MOMENT CAPACITY
(Refer IRC:24-2010 Sections 509)

Fig. 44 Load Path Detailing Considerations for Torsional Moment Transfer (Bearing Type Bolts)

Certain combinations of forces result in similar load effects on individual elements, for example, axial force and major axis moment, or weak axis shear and torsion. If such forces are transferred using flange plate connections, they may add to one another at one flange, and oppose one another at the other flange. If there is possibility for reversal in direction of force, both flanges must be designed for the worst case scenario (load effects are added).

Finally, combined force effects may result in a common element resisting forces in two different directions. This is the case when web plates are used to transfer major axis shear plus major axis moment (or axial force). Another example of such a situation is a connection detail where flange plates are used to transfer major axis moments as well as minor axis shear (or torsion). In such cases, the relevant elements (plates, bolts etc.) must be designed for stress interaction per design standards.

4.1.5 Do's and Don'ts of Bolted Connections

i) It is recommended that, within the same project, bolts of identical sizes should always have identical grades or strengths.
ii) If higher strength bolts are required, they should be noticeably different in size from bolts of a different grade.

iii) It is good practice to not include bolts of consecutive sizes in any given project. For example, if M8 bolts are specified in any project, the next higher bolt size should preferably be M12 and not M10.

iv) If friction grip bolts are used, it is necessary to prepare the faying surfaces to achieve the required coefficient of friction (see RDSO BS-111).

v) All bolted connection details should be scrutinized to ensure adequate clearance for bolting access.

vi) In case any welded plate details are designated as "field weld" to provide adequate bolting access, care should be taken to ensure adequate structural stability and stiffness in temporary or erection condition.

vii) Use of different grade of steel should be avoided in similar structural elements in order to avoid inadvertent errors during construction.

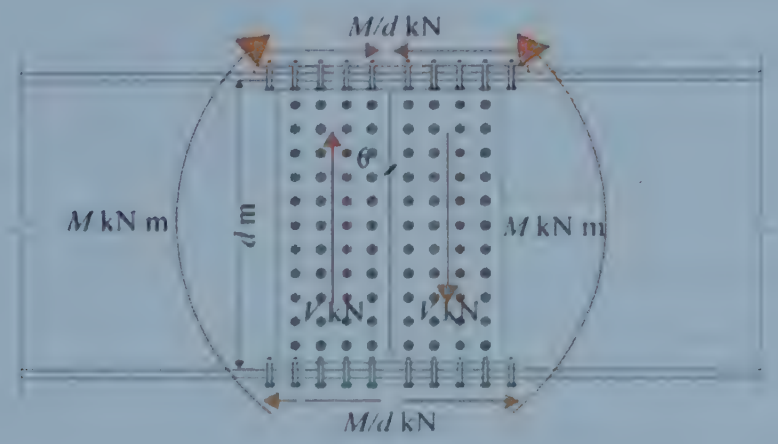

M/d kN
M kN m
d m
M kN m
V kN
V kN
M/d kN
LOAD PATH FOR MAJOR AXIS SHEAR
1. SHEAR FORCE AND BENDING MOMENT INTERACTION CAPACITY AT BEAM CRITICAL NET SECTIONS
(Refer IRC:24-2010 Section 509)
2. BEAM BEARING ON BOLT CAPACITY
(Refer IRC:24-2010 Section 512.5)
3. BOLT DOUBLE SHEAR CAPACITY (IF TWO WEB PLATES ARE PROVIDED, ONE ON EITHER SIDE)
(Refer IRC:24-2010 Section 512.5, see Note 2 on Bolted Shear Connection Detail)
4. BOLT BEARING ON PLATE CAPACITY
(Refer IRC:24-2010 Section 512.5)
5. PLATE NET SECTION SHEAR AND MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)
LOAD PATH FOR MAJOR AXIS MOMENT -- SEE BOLTED MAJOR AXIS MOMENT DETAIL

Fig. 45 Load Path Detailing Considerations for Major Axis Moment + Shear Transfer in Bolted Splice Connection

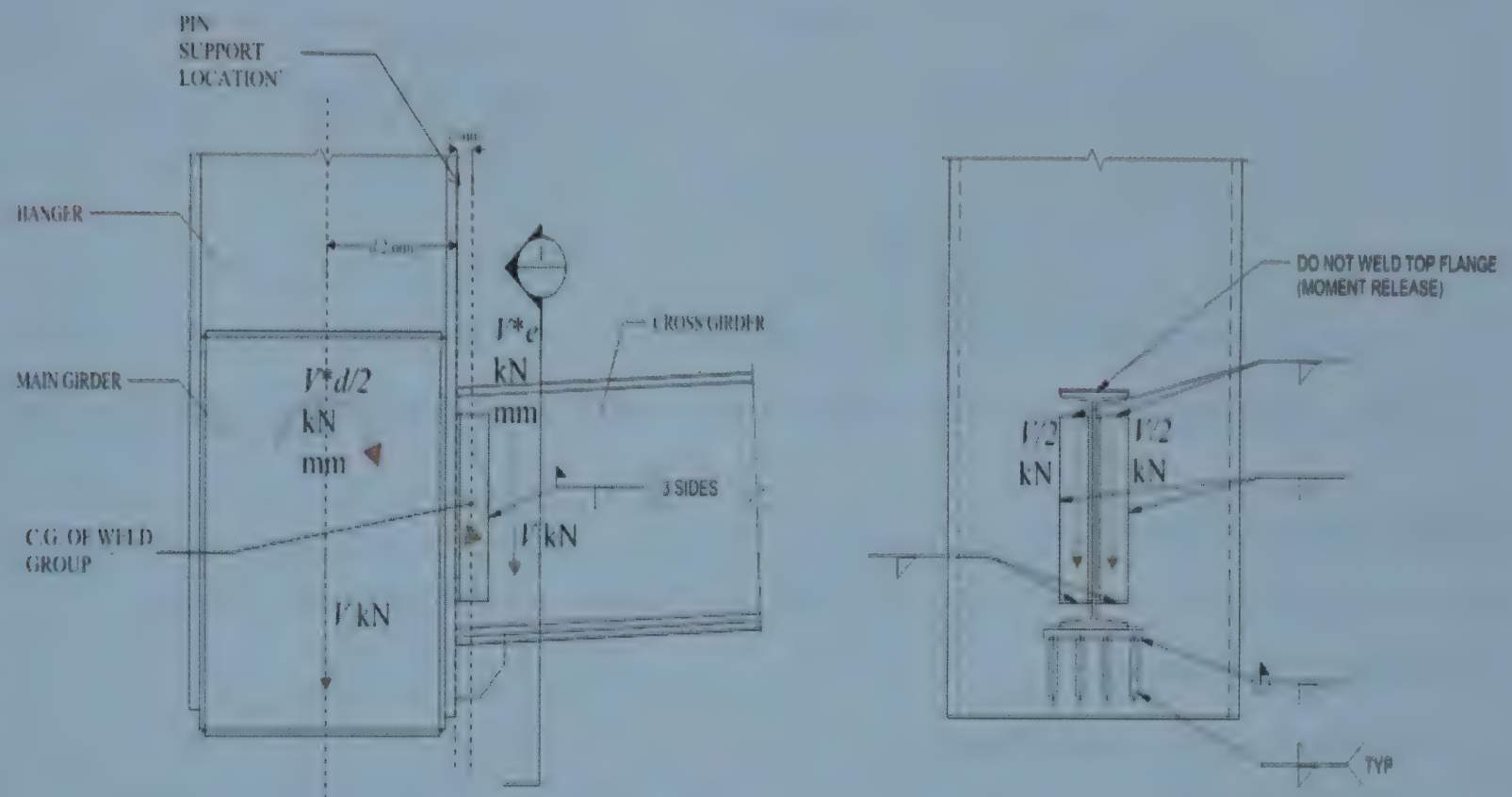
4.2 Welded Connections-General Detailing

General detailing, conventions, symbols, and terminology for welded connections are discussed in **Section 3**.

4.2.1 Detailing for Shear Force

i) Detailing for Major Axis Shear Force

Examples of major axis shear connections are provided in 4.1.1. An example load path for major axis shear force transfer through a welded connection detail is shown in **Fig. 46**. As stated before, as- detailed condition should always be taken into account during modelling and connection design. For example, in the shear connection shown in **Fig. 46**, partial fixity may result in bending stresses if the member cannot rotate. It should be ensured that this partial fixity does not lead to cracking in the weld, member or plate.



AT SUPPORTED MEMBER (CROSS GIRDER)

1. SHEAR FORCE AND BENDING MOMENT CAPACITY AT CRITICAL SECTION
(Refer IRC:24-2010 Section 509)

2. ECCENTRICALLY LOADED WELD GROUP SHEAR CAPACITY
(Refer IRC:24-2010 Section 512.4)

3. PLATE SECTION SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)

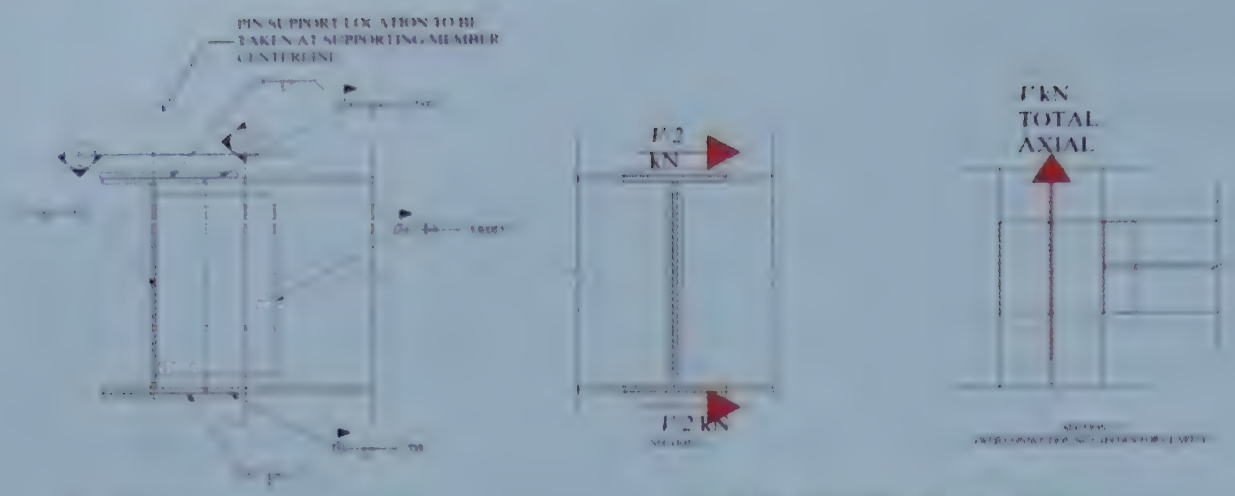
NOTES:
*1) SEE NOTE 1 ON BOLTED SHEAR CONNECTION DETAIL

2) SEE NOTE 3 ON BOLTED SHEAR CONNECTION DETAIL

Fig. 46 Load Path Detailing Considerations for Welded Major Axis Shear Transfer

ii) Detailing for Minor Axis Shear Force

Examples of minor axis shear force resisting locations are discussed in 4.1.1(2). An example load path for minor axis shear force transfer in a welded joint is described in **Fig. 47**.



AT SUPPORTED MEMBER

1. WEAK AXIS SHEAR FORCE CAPACITY FOR MEMBER
(Refer IRC:24-2010 Section 509 considering minor axis shear area)

2. ECCENTRICALLY LOADED WELD GROUP CAPACITY
(Refer IRC:24-2010 Section 512.4)

3. PLATE SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)

AT SUPPORTING MEMBER

11. WELD GROUP SHEAR CAPACITY
(Refer IRC:24-2010 Section 512.4)

12. SUPPORTING MEMBER SECTION AXIAL CAPACITY
(Refer IRC:24-2010 Sections 506 and 507)

Fig. 47 Load Path Detailing Considerations for Welded Minor Axis Shear Transfer

4.2.2 Detailing for Axial Force

Examples of locations where axial force is to be transferred are discussed in **4.1.2**.

4.2.3 Detailing for Moment

i) Detailing for Major Axis Moment

An example load path for transferring major axis moment in a welded splice joint is described in **Fig. 48**.

LOAD PATH FOR MAJOR AXIS MOMENT

1. MAJOR AXIS MOMENT CAPACITY FOR MEMBER
(Refer IRC:24-2010 Section 509)

2. FILLET WELD SHEAR CAPACITY BETWEEN BEAM AND FLANGE PLATE
(Refer IRC:24-2010 Section 512.4)

3. FLANGE PLATE TENSION AND COMPRESSION CAPACITY
(Refer IRC:24-2010 Sections 506 and 507)

Fig. 48 Load Path Detailing Considerations for Welded Major Axis Moment Transfer

ii) Detailing for Torsion

Example locations where torsional loads may arise are discussed in 4.1.3 (2). An example load path for transferring major axis moment in a welded joint is described in **Fig. 49**.

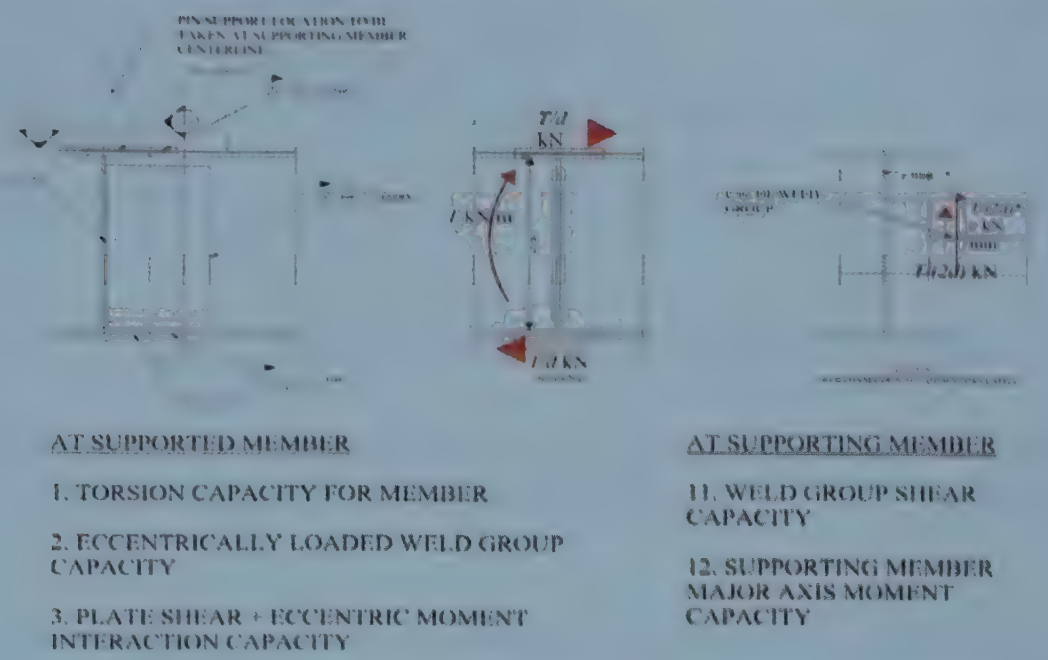

AT SUPPORTED MEMBER
1. TORSION CAPACITY FOR MEMBER
2. ECCENTRICALLY LOADED WELD GROUP CAPACITY
3. PLATE SHEAR + ECCENTRIC MOMENT INTERACTION CAPACITY
AT SUPPORTING MEMBER
11. WELD GROUP SHEAR CAPACITY
12. SUPPORTING MEMBER MAJOR AXIS MOMENT CAPACITY

Fig. 49 Load Path Detailing Considerations for Welded Torsion Transfer

4.2.4 Detailing for Combined Effect

Detailing considerations for combined load effects are discussed in **4.1.4**. An example load path for a welded splice joint transferring major axis moment and major axis shear force is shown in **Fig. 50**.

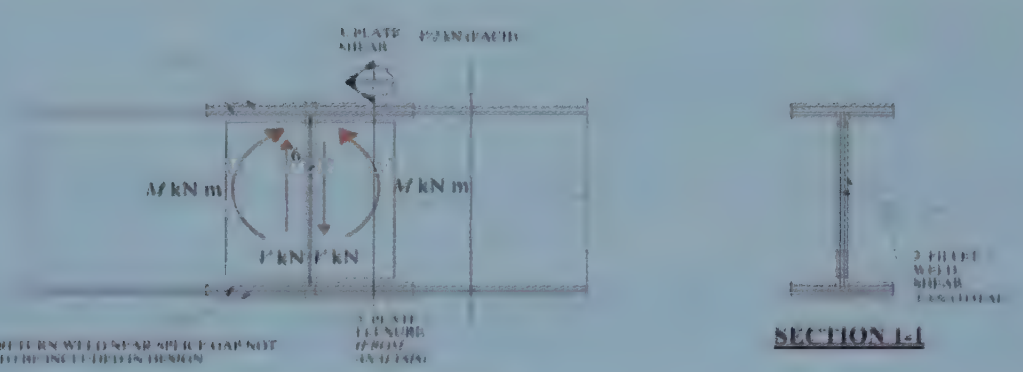

M kN m
M kN m
P kN
P kN
SECTION 1-1
LOAD PATH FOR MAJOR AXIS SHEAR
1. SHEAR FORCE AND BENDING MOMENT INTERACTION CAPACITY AT BEAM SECTION
(Refer IRC:24-2010 Section 510)
2. FILLET WELD SHEAR CAPACITY BETWEEN BEAM AND WEB PLATE
(Refer IRC:24-2010 Section 512.4)
3. PLATE MAJOR AXIS SHEAR + MOMENT INTERACTION CAPACITY
(Refer IRC:24-2010 Section 510)
LOAD PATH FOR MAJOR AXIS MOMENT -- SEE WELDED SPLICE MAJOR AXIS MOMENT LOAD DETAIL

Fig. 50 Load Path Detailing Considerations for Major Axis Moment + Shear Transfer in Welded Splice Connection

4.2.5 Do's and Don'ts of Welded Connections

i) Welded structures should never be modified (through member removal) without proper stu of forces and stresses which are already locked into the original structure.

ii) End returns for fillet welds should be provided as per Clause 512.4.4.1 of IRC:24 and effecti length of fillet welds is to be calculated as per Clause 512.4.3.2 of IRC:24.

iii) Fillet weld lines should preferably be loaded along their lengths in shear, and not in tensio across their throats.

iv) For weld lines loaded transversely, groove welds are preferable.

v) Overhead welding is to be avoided as much as possible. If it is unavoidable, proper electroc should be specified that is appropriate for this application (**Refer Section 3 of th Guidelines**).

vi) Drawings should specify welding consumables (for example, electrodes) (**Refer Section 3 this Guidelines**).

vii) In case of flare-bevel welds, detailing should indicate surface edge bevel preparation.

viii) Refer to **Section 3** for precautions to be taken during the welding process.

4.3 Bearing Connections

Bearing connections are found at the interface of girder and pier cap.

The following points should be kept in mind during the detailing of bearing connections:

i) There should be compatibility between steel design and bearing design. For this reaso steel detailer needs to coordinate with bearing manufacturer at early stages of any project.

ii) Jacking locations need to be identified and specified on the drawings at the start of ar project. There should be adequate structural stiffness to support jacking at these location For this reason, appropriate structural members and stiffeners need to be provide Generally, in plate girder bridges, these are provided on the end cross-girders as shown **Fig. 51**.

iii) Stiffeners should be provided to prevent local instability failures of cross-sectional elements.

iv) Bearing stiffeners should be sufficient in number to cover the total longitudinal movement bearing point. Proper load transfer from the bottom flange to the bearing stiffener should b ensured.

v) Bearing should attach to girders through bolted connections (rather than welded connection to ensure ease of replacement.

vi) Packing plates may be provided to account for non-horizontal girder soffits.

C L OF JACK
C L OF JACK
C L OF JACK
C L OF JACK
C L OF JACK
C L OF JACK
END DIAPHRAGM (TYP)
INTERMEDIATE CROSS FRAME (TYP)
PLATE GIRDER (TYP)
C L OF PIER C L OF BEARING

Fig. 51 Typical Plate Girder Bridge Plan Identifying Jacking Locations on End Diaphragm

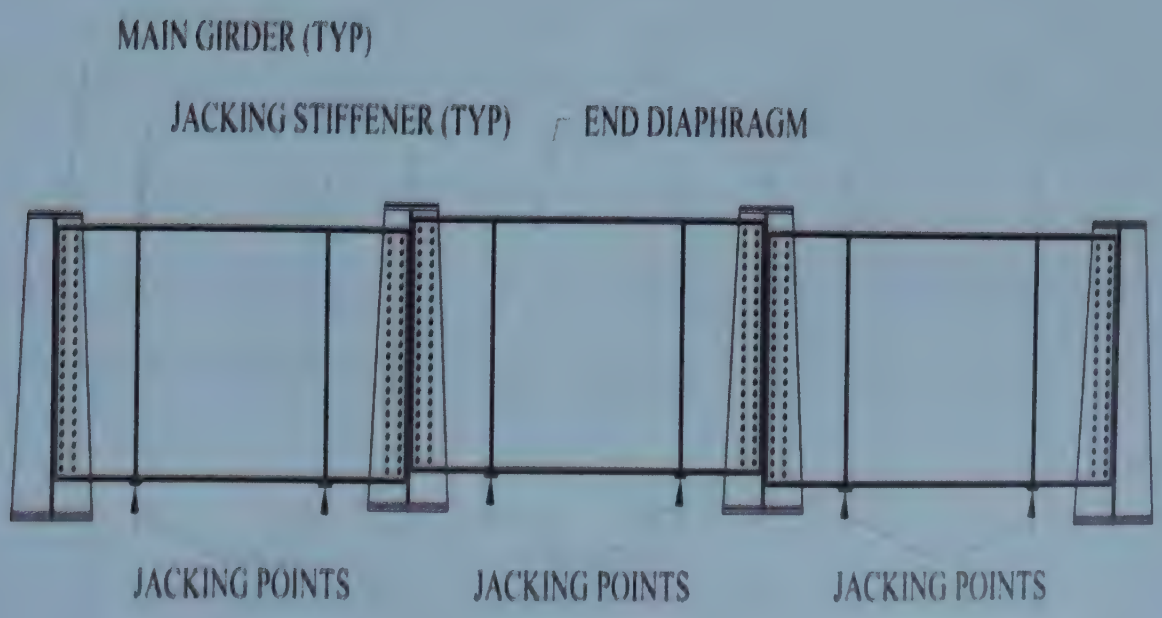


Fig. 52 Section A-A on Jacking Locations Plan Showing Jacking Points and Jacking Stiffeners

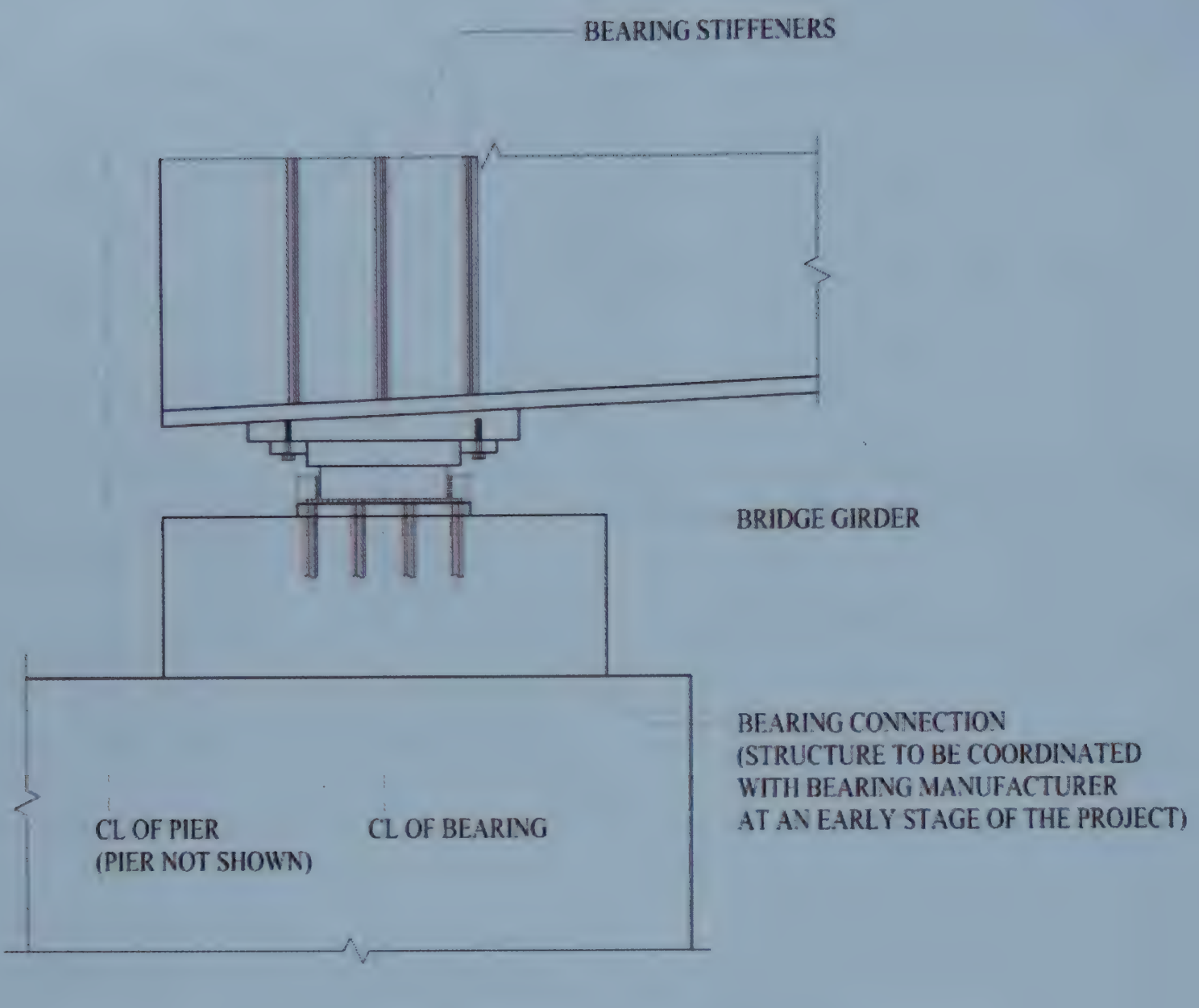


Fig. 53 Typical Bearing Connection Detail

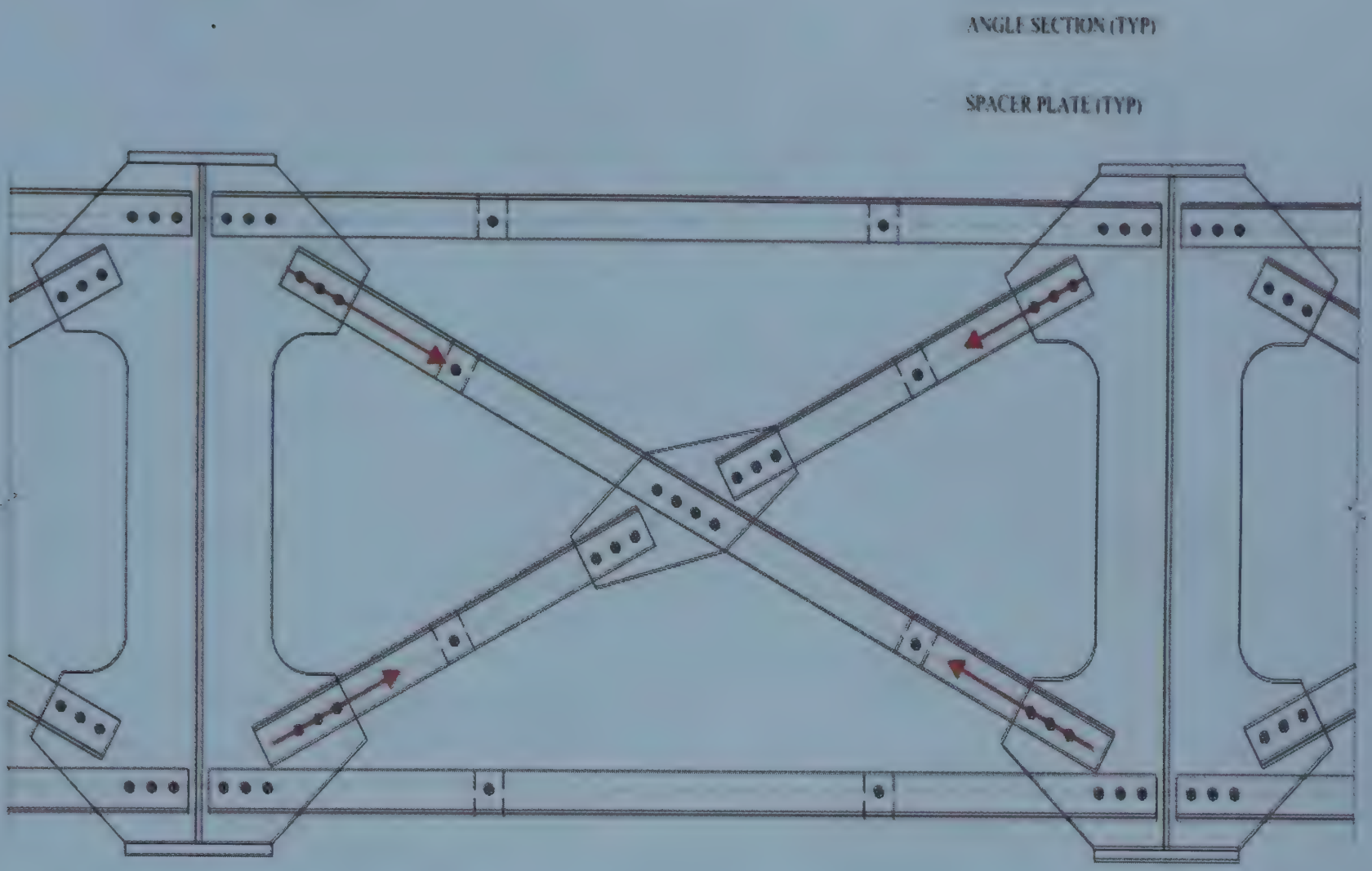
4.4 Stiffeners

Stiffeners in I-shaped cross sections are necessary to prevent local distortion of the cross-section, for example, due to web local buckling or flange bending. This should particularly be considered at any local load transfer points. Examples of stiffeners include, but are not limited to, bearing stiffener, jacking stiffener, moment and axial connection stiffeners and stiffeners around opening.

Aside from concentrated force locations, stiffeners are also provided in girders with slender webs. Girder webs designed using simple Post-critical method as per Clause 509.4.2.2 (a) of IRC 24 require transverse stiffeners at the supports. Girder webs designed using tension field method as per Clause 509.4.2.2 (b) of IRC 24 require intermediate transverse stiffeners in addition to end stiffeners.

4.5 Bracing Members

Bracing members are used to reduce unbraced length of members by providing lateral restraint, and to develop diaphragm action to resist lateral loads. Commonly used bracing configurations include cross or triangular bracing, K-type or V bracing and beam type bracing. Typical load path detailing considerations are shown for cross-bracing in plate girders in **Fig. 54**. Corresponding considerations may be made if a different bracing configuration (V-type or beam bracing) in used.



LOAD PATH FOR AXIAL FORCE CONNECTION

(1) AXIAL COMPRESSION CAPACITY CONSIDERING UNBRACED LENGTH OF BRACE

(2) NET TENSILE RUPTURE AT CRITICAL SECTION OF BRACE

(3) BRACE BEARING ON BOLT CAPACITY

(4) NET TENSILE RUPTURE OF GUSSET PLATE

(5) YIELDING OF WHITMORE SECTION OF GUSSET PLATE

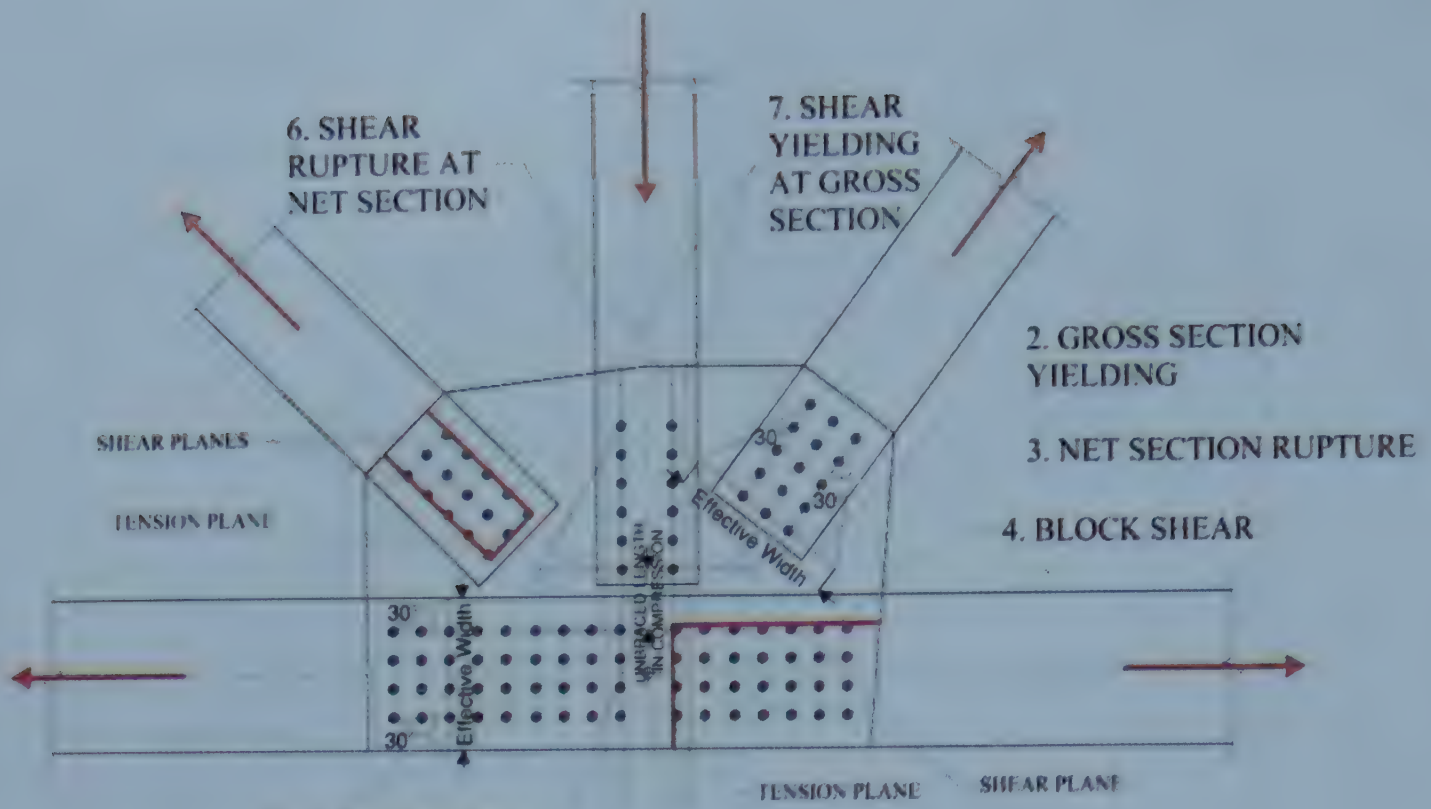
(6) GUSSET PLATE LOCAL BUCKLING

(7) FLEXURE DUE TO ECCENTRIC OR UNBALANCED MOMENT AT GUSSET PLATE

Fig. 54 Load Path Detailing Considerations for Diaphragm Bracing Members

4.6 Gusset Plates

Gusset plates are found at intersections of axial force carrying members. As a result, they are subjected to forces acting along the axes of such members. The direction in which these forces act may be reversible (that is, both directions need to be considered during design) if they result from wind or earthquakes. Load path detailing considerations for gusset plates is described in **Fig. 55**.



ALL FORCES TO INTERSECT AT COMMON WORK POINT

AT GUSSET PLATE

1. BOLT BEARING
(Refer IRC:24-2010 Section 512.5)

2. GROSS SECTION YIELDING
(Refer IRC:24-2010 Clause 506.1.1)

3. NET SECTION RUTURE INCLUDING SHEAR LAG EFFECTS
(Refer IRC:24-2010 Clause 506.1.2)

4. BLOCK SHEAR
(Refer IRC:24-2010 Clause 506.1.3)

5. COMPRESSION
(Refer IRC IRC:24-2010 Section 507)

6 and 7. SHEAR DESIGN
(Refer IRC:24-2010 Clause 509.4)

Fig. 55 Gusset Plate Load Path Detailing

Gusset plates should be designed such that all forces meet at a common work point as shown in **Fig. 55**. If the forces do not meet at a common point, they will create an unbalanced moment on the gusset plate.

In addition to the ultimate limit states listed on **Fig. 55**, gusset plates should also be checked for stress interaction limit states under combined loads and/or other serviceability considerations. Stress interaction limit states include the interaction of axial, shear and flexural stresses as well as principal and Von Mises stresses at critical sections. Fatigue on gusset plates should be checked per Clause 511 of IRC:24. Examples of unbraced length of gusset plate to be used for checking compression are shown in **Fig. 56**.

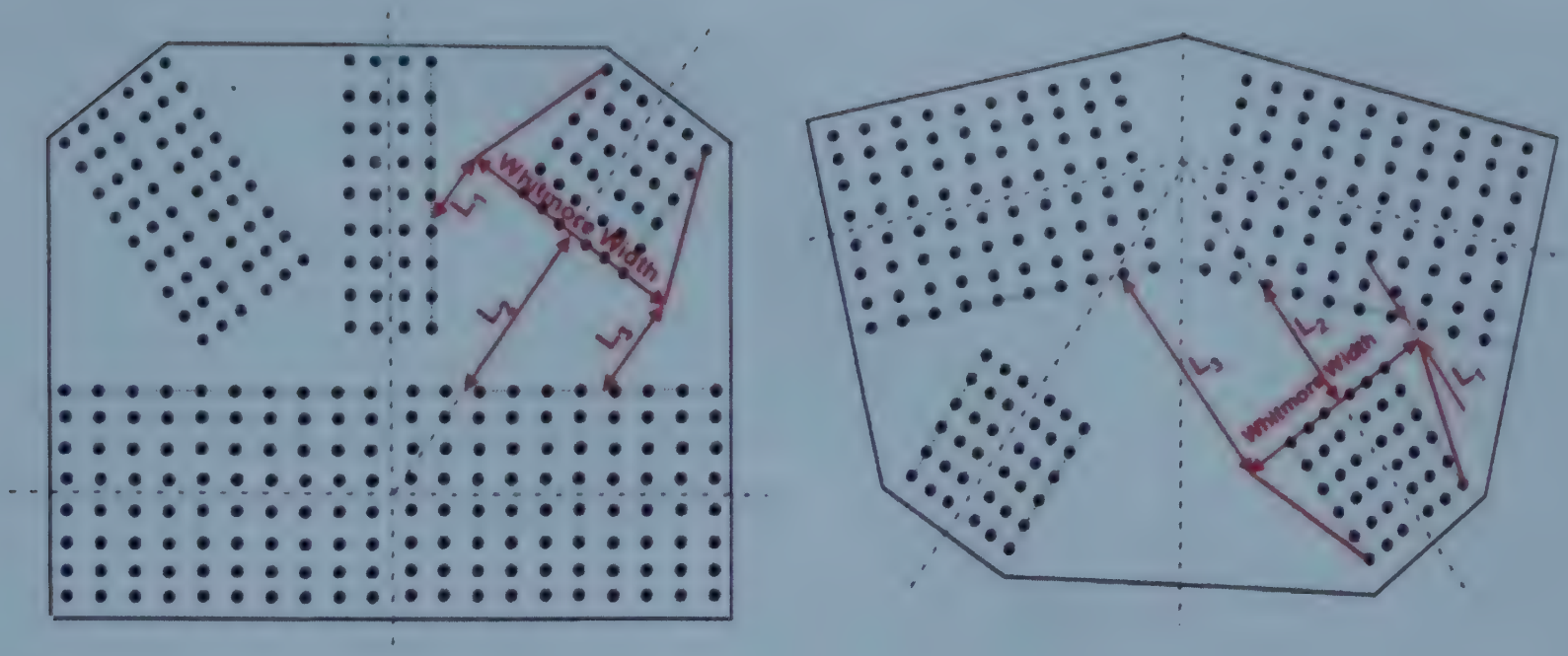


Fig. 56 Gusset Plate Unbraced Length Examples Unbraced Length shall be Taken as the Largest Among L1, L2 and L3

Joint block may also be configured in a way such that splices are provided on the members, as an alternate to a gusset connection, as shown in **Fig. 57**.

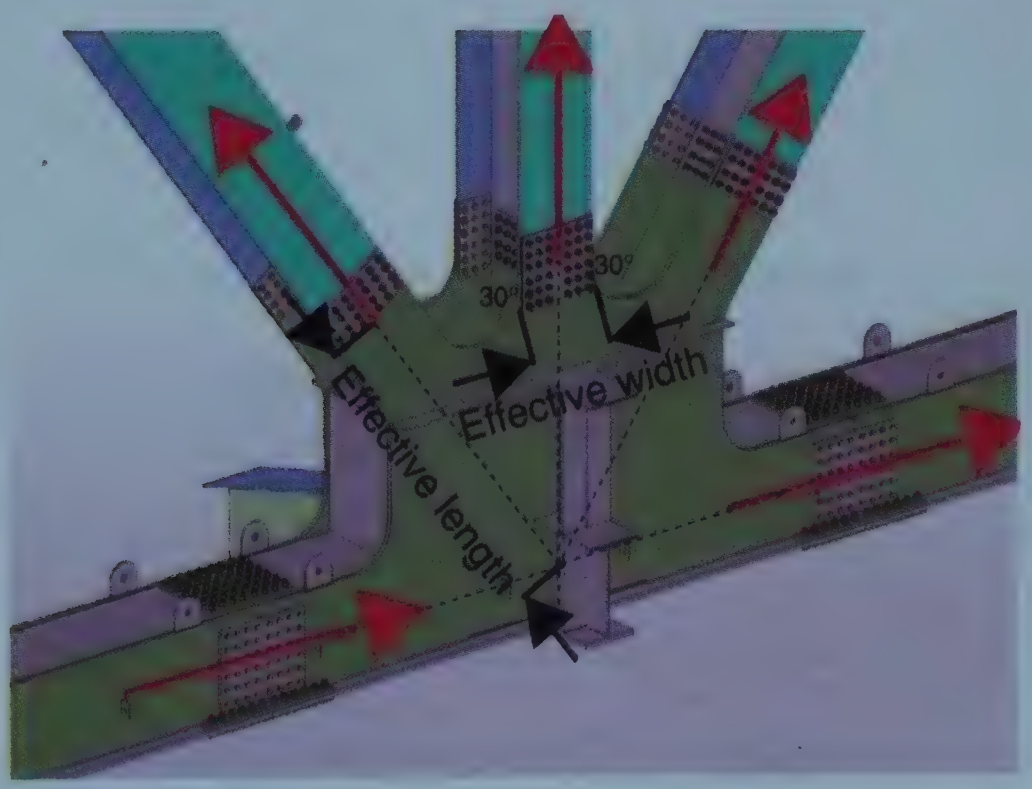


Fig. 57 Gusset Plate Detail with Splices on Members, Force Directions and Dimensions are for an Example Scenario

4.7 Shear Connectors

Shear connectors are provided for horizontal shear transfer to develop composite action between steel girder and concrete deck. Design drawings should clearly specify shear connector location/ spacing, welding techni;que, size, material, shape of tip and testing requirements. Shear connection location / spacing to be specified on design drawings. A typical shear connector detail is shown in **Fig. 58**.

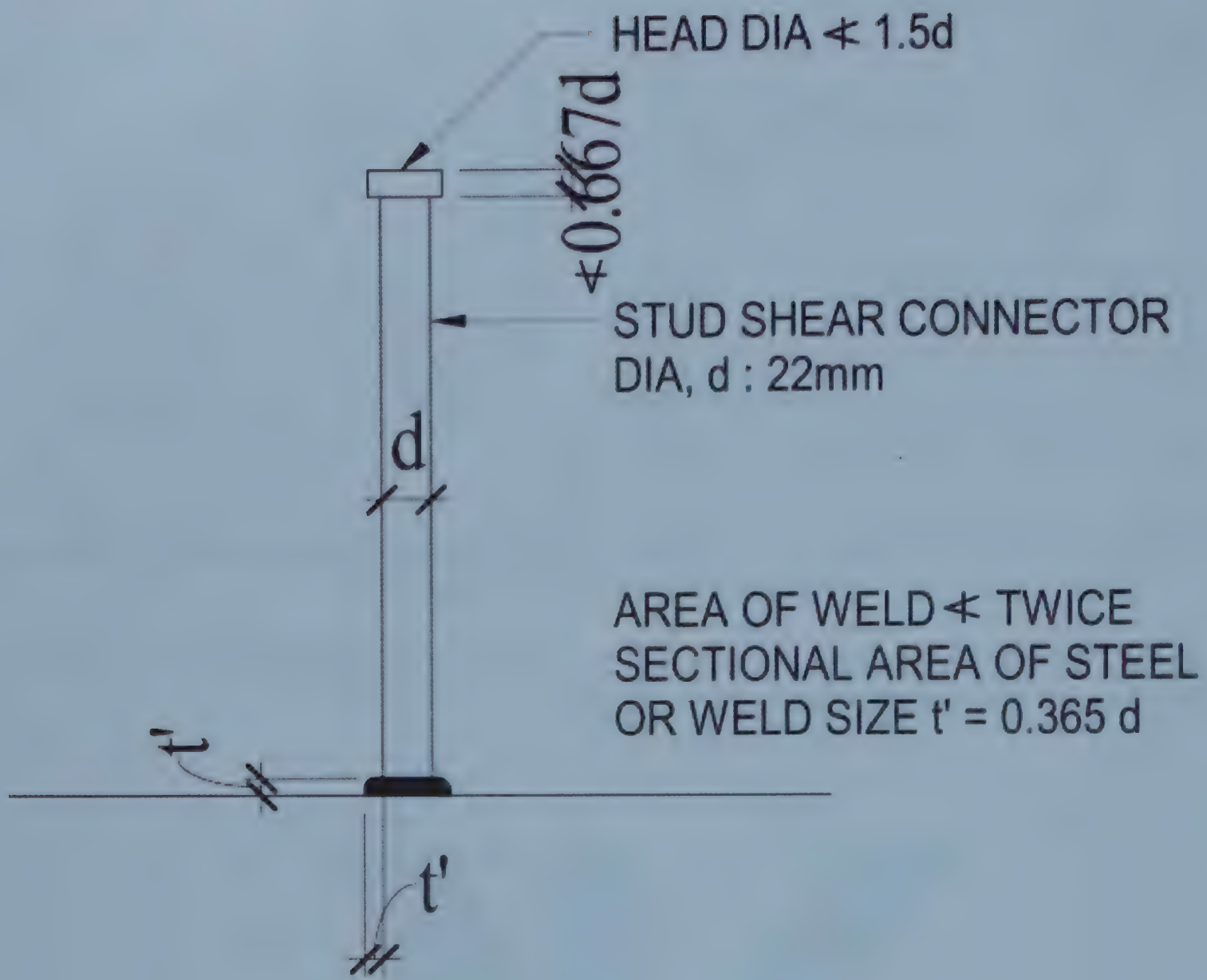


Fig. 58 Typical Shear Connector Detail

5 STEEL BRIDGE DETAILING

This section highlights some typical detailing feature of standard steel bridge types i.e., plate girder bridges (5.1) & open web girders (5.2).

The typical detailing features of bolts and welds is explained in detail in **Section 3**.

The force transfer mechanism of typical connection is highlighted in **Section 4** of this guidelines.

5.1 Plate Girder Bridges

The design of plate girders shall be as per the relevant Clause of IRC:24. The design of composite plate girders shall be as per the relevant Clause of IRC:22, IRC:24 & IRC:112. The minimum thicknesses of plates and rolled sections shall be as per Clause 504.7 of IRC:24. The fatigue consideration should be in accordance with the recommendations of Clause 511 of IRC:24. Ease of fabrication and erection, as also subsequent inspection and maintenance should be considered while detailing the connections. The detailing provisions of Clause 504.9 of IRC:24 shall be kept in mind while designing & detailing bearings.

A typical steel concrete composite plate girder bridge consists of the following components:

5.1.1 Longitudinal Girder

i) The spacing of girders shall be as per Clause 504.3 of IRC:24.
ii) The depth of girders shall be as per Clause 504.4 of IRC:24.
iii) The provisions of camber (as applicable) shall be as per Clause 504.6 of IRC:24.
iv) The minimum web thickness shall be as per Clause 504.9.6.1 of IRC:24.
v) The clear dimension of a web panel shall be as per Clause 509.7.14, g) of IRC:24.

5.1.2 Cross Frames/Bracings (as, applicable)

i) The spacing of the cross frames/bracings is determined by the lateral forces to be transferred between panels and the lateral torsional buckling requirement.
ii) The lateral bracings shall be as per Clause 509.8 of IRC:24.

5.1.3 Cross Girder/Diaphragm

i) End
ii) Intermediate (as, applicable)

5.1.4 Stiffeners

i) Intermediate transverse web stiffener:

- The outstand of stiffener shall be as per Clause 509.7.1.2 of IRC:24.
- The spacing of stiffeners shall be as per Clause 509.7.2.2 of IRC:24.
- The minimum moment of inertia of stiffeners shall be as per Clause 509.7.2.4 of IRC:24.
- The connection of stiffeners to web shall be as per Clause 509.7.2.6 of IRC:24.

ii) Load carrying stiffener:

- Connection of load carrying stiffener to web shall be as per Clause 509.37.10 of IRC:24.
- They are usually provided at cross frame/bracing location and at cross girder location.

iii) Bearing stiffener:

- Bearing stiffeners should project nearly as much as the overhang of the flange through which load is transferred.
- Connection of bearing stiffener to web shall be as per Clause 509.37.10 of IRC:24.
- They are usually provided at bearing location and at jacking location.

iv) Connection to flanges:

- Connection of flanges in tension shall be as per Clause 509.7.11.1 of IRC:24.
- Connection of flanges in tension shall be as per Clause 509.7.11.2 of IRC:24.

v) Longitudinal stiffeners:

- The longitudinal stiffeners shall be detailed as per Clause 509.7.13 of IRC:24.

vi) Other detailing requirements:

- The detailing of stiffener shall be as per Clause 509.7.14 of IRC:24.

Note:

i) The same stiffeners may perform more than one function and their design should comply with the requirements of all the functions designed for.

ii) There may be other type of stiffeners like diagonal stiffener, tension stiffener & torsion stiffener in a bridge depending upon the design & detailing methodology adopted by the designer.

iii) Stiffeners not subject to external loads or moments may terminate clear of the tension flange and in such a situation the distance cut short from the line of the weld should not be more than $4t_w$.

5.1.5 Shear Connectors

i) The detailing of shear connector shall be as per Clause 606.6 of IRC:22.

ii) The limiting criteria for spacing of shear connector shall be as per Clause 606.9 of IRC:22.

5.1.6 Seismic Stoppers (as, applicable)

5.1.7 Connections

i) The connections shall be detailed as per Clause 512 of IRC:24. Some key features of the same are:

- The alignment of member shall be as per Clause 512.3 of IRC:24.
- General requirements of welds shall be as per Clause 512.4.4 of IRC:24.
- Where a fillet weld is applied to the edge of a plate or section, provisions of Clause 512.4.6 of IRC:24 shall be followed.
- Connections made with Bearing Type Bolts, Rivets or Pins shall be as per Clause 512.5 of IRC:24. Minimum dia. of fasteners used in load bearing members shall be 16 mm diameter.

ii) Typical connection in a plate girder bridges are:

- **Splice connections** The location of this connection is decided based on the following factors:
  - availability of plate size/length,
  - transportation constraints
  - design requirements

Note: *They should ideally be placed as far as possible away from the sections of maximum forces.*

- Connection between web & flange
- Connection between web / flange & stiffener

5.1.8 Deck Slab

i) The detailing of haunches shall be as per Clause 606.7 of IRC:22.

ii) The cover to shear connector shall be as per Clause 606.8 of IRC:22.

The durability provisions for plate girder shall be as per Annex D of IRC:24. The durability provision of deck slab shall be as per Clause 14 of IRC:112.

Appendix – 3 shows a typical general arrangement of a plate girder bridge and shows how to detail each component with cross reference to **Section 3 & 4**. It may be noted that this is only a reference drawing, and the designer is free to adopt any other detailing as per their design methodology.

5.2 Open Web Girder Bridge

The design of open web girders shall be as per the relevant Clause of IRC:24. The design of composite plate girders shall be as per the relevant Clause of IRC:22 & IRC:24.

Usually, an open web girder consists of a main truss which is then connected to a series of cross girders, stringers, and bracings. The design and detail of the cross girders, stringers and bracings is similar to that of a plate girder / beam. However, the truss elements and their gusset connection are different from plate girder / beam and require specific attention. Therefore, the provisions of plate girders as detailed in 5.1 are applicable to this section as well.

A typical open web girder bridge consists of the following components:

5.2.1 Truss Members: shall be detailed as per Clause 506 & 507 (as applicable) of IRC:24.

i) Top chord
ii) Bottom chord
iii) Verticals
iv) Diagonals

5.2.2 Cross Girders: Refer 5.1 (3)

5.2.3 Stringers: Refer 5.1 (1)

5.2.4 Bracings (as, applicable):

i) The lateral bracings shall be detailed as per Clause 508.11 of IRC:24.
ii) The portal bracings shall be detailed as per Clause 508.11.6 of IRC:24.
iii) The sway bracings shall be detailed as per Clause 508.11.5 of IRC:24.

5.2.5 Lacing / Batten (as, applicable)

i) Lacings:
- In tension members shall be detailed as per provisions of Clause 506.2.7 of IRC:24.
- In compression members shall be detailed as per provisions of Clause 507.8 of IRC:24.

ii) Battens:
- In tension members shall be detailed as per provisions of Clause 506.2.8 of IRC:24.
- In compression members shall be detailed as per provisions of Clause 507.9 of IRC:24.

5.2.6 Stiffeners: Refer 5.1 (4)

5.2.7 Shear Connectors: Refer 5.1 (5)

5.2.8 Seismic Stoppers (as, applicable)

5.2.9 Connections

i) Gusset Connections: The gusset connections shall be detailed as per Clause 508.6 & 512.5.3 of IRC:24.
ii) All others as per 5.1 (7).
iii) All others as per 5.1 (7).

Appendix – 4, shows the general arrangement of an open web girder bridge and shows how to detail each component with cross reference to **Section 3 & 4**. It may be noted that this is only a reference drawing, and the designer is free to adopt any other detailing as per their design methodology.

6 QUALITY ASSURANCE AND QUALITY CONTROL

6.1 Quality Assurance and Quality Control

Quality Assurance (QA) and Quality Control (QC) are two terms of quality management that are often used interchangeably. Although similar, there are distinct differences between the two concepts.

Quality Assurance (QA) is an ongoing management responsibility that involves planning and implementing systematic actions to assure that the work processes and designed elements are functioning as intended. A properly implemented QA program will eliminate construction delays.

Quality Control (QC) is the system used by the Designer/ Contractor/ Fabricator to monitor, assess, and adjust their production or placement processes to ensure that the final product will meet the specified level of quality.

QA and QC are important for project implementation during design, off-site fabrication, field erection as well as repair. It is important that the stakeholders of the project adopt and implement a standard in the form of a Quality Assurance Plan for implementing the QA/QC process.

QA /QC is applicable to all Stages of Project such as:

i) Planning, Design and Drawing
ii) Documents for shop Fabrication
iii) Transportation, erection, and Final joining (Bolting/Welding) at site

6.2 Planning Design and Drawing

6.2.1 Planning

At the planning stage detailing of the following shall be ensured:

i) The layout of the bridge to ascertain the requirement of straight /curved girders and requirement of super elevation, if any.
ii) The type of superstructure and span length required for the existing site condition.
iii) Approach to site to decide the maximum size of components.
iv) The area in the vicinity of the bridge for movement of crane for handling and erection.
v) Safety aspect with respect to movement of traffic at the time of erection.
vi) Presence of overhead lines and other obstructions.

6.2.2 Design and Drawing

i) Selection of raw material size, preferably, in the available rolling sizes from the plants.
ii) Selection of joints to ensure maximum dimension of components for transportation and handling and erection (as specified in planning).
iii) The type of bearings, the load chart for designing the bearings, the chart showing the position of different types of the bearing on the pedestals.
iv) Position of jacking points and the clearance from pier cap.
v) The position for attaching lifting tackles on the component for handling and erection.
vi) Camber diagram and lengths of component (Shortening / Increasing of components as compared to geometrical dimensions) to achieve the camber.
vii) Requirement for finishing for butting face of the field joints (Hand ground / End milled surface).
viii) Method to make the curving in girders (Heat curved / Cutting of flanges), if required.
ix) Shop weld joint type (Fillet, Butt).
x) Showing the sizes of weld in joints with proper weld symbols and mentioning the welding process.
xi) Welding access for execution of the joint.
xii) Requirement for types of weld tests for different weld joints (DPT, MPT, UT, RT).
xiii) Types of surface preparation and painting schedule for the existing environmental condition at site.
xiv) The surface treatment of mating surfaces under the splice joints of the girder.

6.3 Transportation, Erection and Field Joining

i) Sequence of loading of components, proper docking and restraining for safety during transportation.
ii) Sequence of erection of the components so as to avoid infringement with crane boom and obstruction of crane movement for handling and placement of the adjoining components.
iii) Load on each staging, type of staging for the load and size of the foundation for the load dispersion.
iv) Planning for height of staging, after making considerations for height of packing and camber jacks, for reaching the girder bottom for support.
v) Maximum radius of operation with respect to heaviest component with the load capacity of crane.
vi) Type of camber jack and its capacity.
vii) Sequence of lifting of girders for placement on bearings and capacity of lifting jacks.
viii) Details of type of platform / arrangement, for workmen, for execution of field joints.
ix) Details of arrangement for protection against wind and rain for execution of welded field joints
x) Details for life line support for movement for work men.
xi) Details of safety arrangements for work men and general traffic, under the girder, against falling debris / tools and tackles.
xii) Protection against power lines in the girder vicinity.
xiii) Bolted field joint type (Turned bolt / Load bearing/HSFG bolting).
xiv) Access to the joint for execution of the bolted field joint and the sequence of joint execution.
xv) Field welding joint type, welding symbol with welding process, sequence of welding and welding access.
xvi) Erection support releasing sequence after completion of site joining.
xvii) Sequence of releasing the bearing restraints.

Note:

IRC:SP:104 "Guidelines for Fabrication and Erection of Steel Bridges" has description and requirements of different aspects of execution such as shop fabrication, testing of welds, dimension of girders, HSFG bolting, trial assembly, painting, site erection etc.

7 REFERENCES

1 IRC:22 "Standard Specifications and Code of Practice for Road Bridges, Section VI Composite Construction (Limit State Design)", Indian Roads Congress

2 IRC:24 "Standard Specifications and Code of Practice for Road Bridges, Section V Steel Road Bridges (Limit State Method)", Indian Roads Congress

3. IRC:SP:104 "Guidelines for Fabrication and Erection of Steel Bridges", Indian Roads Congress

4 IS: 800-2007 General Construction in Steel - Code of Practice, Bureau of Indian Standards

5 Research Design and Standards Organization - Report no BS-111 ,Guidelines for use of High Strength Friction Grip (HSFG) bolts on bridges in Indian Railways

6 American Institute of Steel Construction - AISC Steel Construction Handbook

7 Gupta, S. and Jain, S. (2011) - "Good engineering practices in concept design, analysis & detailing o steel concrete composite bridges" In: The Bridge and Structural Engineer, Volume 41, No. 4

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/ Manual etc. from the date specified therein)

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